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NAVIGATION CONDITIONS AT LOCKS AND DAM 26, MISSISSIPPI RIVER; H--ETC(U)
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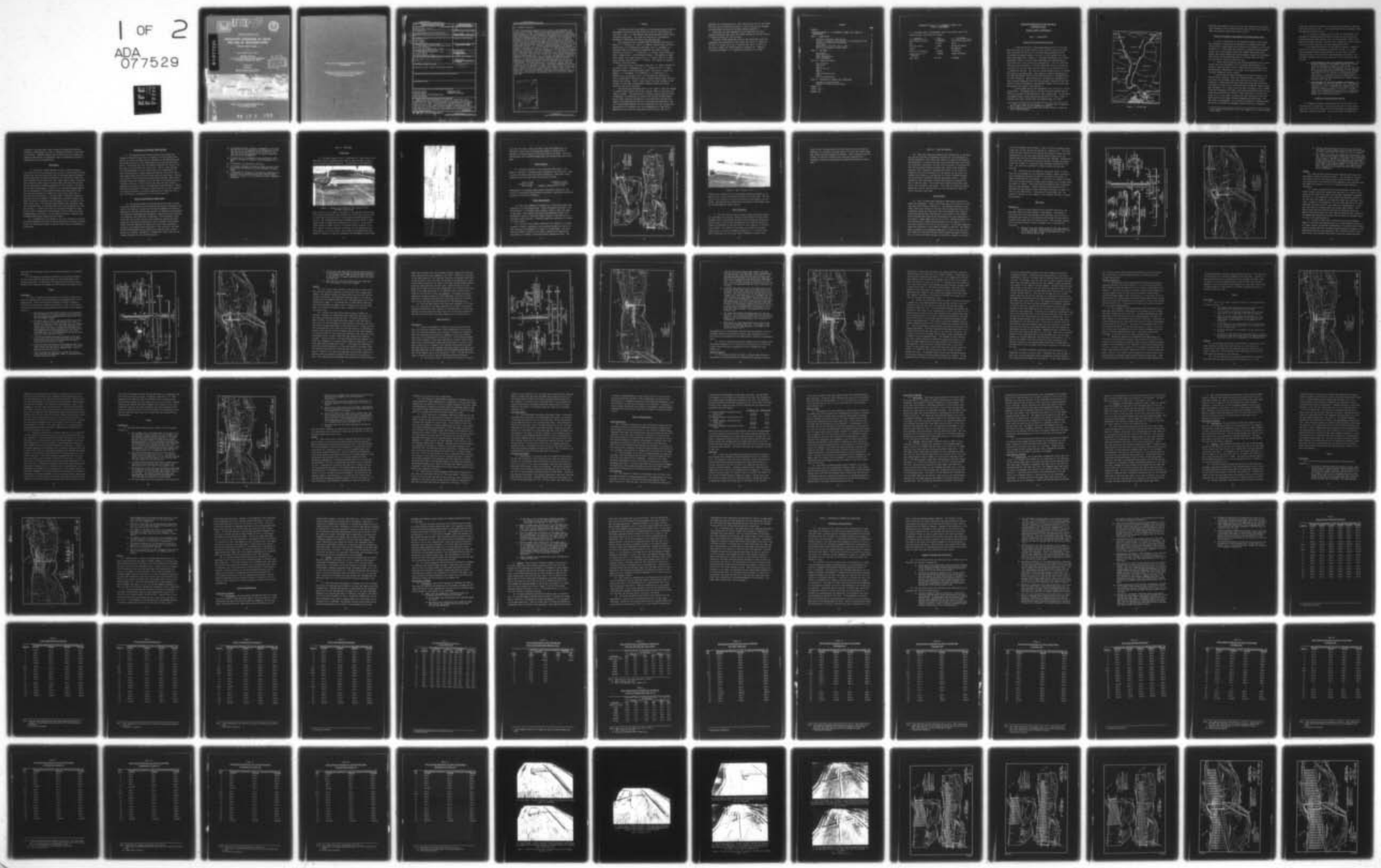

TECHNICAL REPORT HL-79-19

NAVIGATION CONDITIONS AT LOCKS AND DAM 26, MISSISSIPPI RIVER

Hydraulic Model Investigation

by

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Final Report

Prepared for U. S. Army Engineer District, St. Louis
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Locks and Dam 26 Replacement is proposed for construction on the Mississippi River about 2 miles downstream of the existing locks and dam at Alton, Illinois, as part of the program for the modernization of navigation facilities on the river. The project would replace the smaller existing structure with a large, modern structure designed to handle two-way traffic. The proposed structure would include two parallel locks separated by two 110-ft gate bays along the left bank and a dam consisting of a gated spillway and fixed overflow section. (Continued)

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20. ABSTRACT (Continued).

A fixed-bed model reproduced about 6.7 miles of the Mississippi River Channel, Ellis and Maple Islands, and the adjacent overbank areas to an undistorted scale of 1:120. The model study was concerned with the composition and configuration of the dam, arrangement and separation of the locks and lock walls, and navigation conditions in the lock approaches. The investigation has produced the following general results and conclusions: (a) Satisfactory navigation conditions for two-way traffic could be developed at either the existing site or the proposed replacement site by providing gates between the locks. The number of gates in the proposed gated spillway could be reduced to seven without producing any significant changes affecting the movement of a tow approaching the locks. (b) Currents in the upper approach to the landward lock at the alternate site would be affected by the eddy forming along the left overbank. Placing a fill along the overbank to an elevation above normal pool would reduce the effect of the eddy on navigation. (c) Ports will be required in the upper guard walls to reduce the intensity of the crosscurrent near the end of the walls. The riverward lock guard wall should extend farther upstream than the landward guard wall. (d) In the lower lock approaches, navigation conditions for two-way traffic could be improved with guard walls shorter than 1200 ft. (e) With the locks separated, navigation conditions could be developed in both the upstream and downstream approaches with the locks used simultaneously without the operation of one interfering with the operation of the other. (f) During construction of the project, adequate navigation conditions could be maintained by using a three-stage cofferdam with the first stage located on the Missouri side and the third stage along the Illinois side. (g) Scouring of the bed can be expected along the upper corners of the cofferdams on the river side, particularly during long periods of extreme flow conditions. Along the main cofferdam, scouring can be eliminated or minimized with a deflector installed at the upstream corner on the river side of each cofferdam.

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PREFACE

The hydraulic model investigation of the proposed Locks and Dam 26 Replacement study was authorized by DA Form 2544 No. ED8-68 dated 10 July 1967 to the U. S. Army Engineer Waterways Experiment Station (WES), Vicksburg, Mississippi. The study was conducted in the Hydraulics Laboratory of WES during the period July 1967-June 1974.

The investigation was conducted under the general supervision of Messrs. E. P. Fortson, Jr., and H. B. Simmons, retired Chief and Chief, respectively, of the Hydraulics Laboratory, and F. A. Herrmann, Jr., Assistant Chief of the Hydraulics Laboratory; and under the direct supervision of Messrs. J. J. Franco and J. E. Glover, retired Chief and Chief, respectively, of the Waterways Division. The engineer in immediate charge of the model study was Mr. L. J. Shows, Chief of the Navigation Branch, assisted by Mr. R. T. Wooley. This report was prepared by Messrs. Shows and Franco.

During the course of the model study, Mr. J. P. Davis (retired) of the Office, Chief of Engineers, Mr. R. I. Kaufman of the U. S. Army Engineer Division, Lower Mississippi Valley, and COL E. R. Decker and Messrs. A. J. Tiefenbrun, H. M. McKinney, R. C. Armstrong, A. L. Johnson, and C. W. Denzel of the U. S. Army Engineer District, St. Louis, visited WES at different times to observe special model tests and discuss test results. The St. Louis District was kept informed of the progress of the study through monthly progress reports and special reports at the end of each test.

The project plan presented herein of two 1,200-ft locks was changed in October 1978 by Public Law 95-502. Section 102(a) of the public law reads as follows: "The Secretary of the Army, acting through the Chief of Engineers, is authorized to replace locks and dam 26, Mississippi River, Alton, Illinois, and Missouri, by constructing a new dam and a single, one-hundred-and-ten-foot by one-thousand-two-hundred-foot lock at a location approximately two miles downstream from the existing dam, substantially in accordance with the recommendations of the Chief of Engineers in his report on such project dated July 31, 1976, at an

estimated cost of $421,000,000." Also, Section 101(j) of the law reads: "The lock and dam authorized pursuant to section 102 shall be designed and constructed to provide for possible future expansion."

This report has been published in final form since the data obtained during the model tests are useful information and can serve as a resource document for other projects.

Directors of WES during the course of the investigation and the preparation and publication of this report were COL J. R. Oswalt, Jr., CE, COL L. A. Brown, CE, BG E. D. Peixotto, CE, COL G. H. Hilt, CE, COL John L. Cannon, CE, and COL Nelson P. Conover, CE. Technical Directors were Messrs. J. B. Tiffany (retired) and F. R. Brown.

CONTENTS

CONVERSION FACTORS, U. S. CUSTOMARY TO METRIC (SI) UNITS OF MEASUREMENT

U. S. customary units of measurement used in this report can be converted to metric (SI) units as follows:

Multiply	By	To Obtain
cubic feet per second	0.02831685	cubic metres per second
feet	0.3048	metres
feet per second	0.3048	metres per second
inches	25.4	millimetres
miles (U. S. statute)	1.609344	kilometres
square miles (U. S. statute)	2.589988	square kilometres
tons (mass)	907.1847	kilograms

NAVIGATION CONDITIONS AT LOCKS AND DAM 26
MISSISSIPPI RIVER

Hydraulic Model Investigation

PART I: INTRODUCTION

Location of Proposed Locks and Dam

1. Locks and Dam 26 is located approximately 202.9 river miles* above the mouth of the Ohio River at Alton, Illinois, and is the lowermost structure in a series of 26 locks and dams constructed for the authorized slack-water project to provide for 9-ft navigation on the upper Mississippi River (Figure 1). Two major tributaries, the Illinois and Missouri Rivers, flow into the Mississippi River approximately 15 miles above and 8 miles below Alton, respectively. The pool created by Dam 26 at normal elevation of 419.0** extends 38.7 miles up the Mississippi River to Lock and Dam 25 and 80.1 miles up the Illinois River to La Grange Lock and Dam. The Illinois provides waterway access to Chicago and thence to the Great Lakes. The section of the pool extending into the upper Mississippi River provides waterway access to Minneapolis and St. Paul, Minnesota. Without exception, the particular navigation projects discussed emphasize the vital and strategic location of Locks and Dam 26 in the ever-expanding inland waterway system, servicing the upper Mississippi River Basin's vast farm belt.

2. The drainage area of the Mississippi River at Alton is 171,500 square miles. The maximum flow of 570,000 cfs and the minimum flow of 8,000 cfs occurred in June 1858 and in 1948, respectively. In June 1844, the maximum stage in this vicinity was recorded at el 432.4. About a hundred years later, January 1954, the minimum stage of el 390.5

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* A table of factors for converting U. S. customary units of measurement to metric (SI) units is presented on page 3.

** All elevations (el) cited herein are in feet referred to mean sea level, 1929 adjustment.

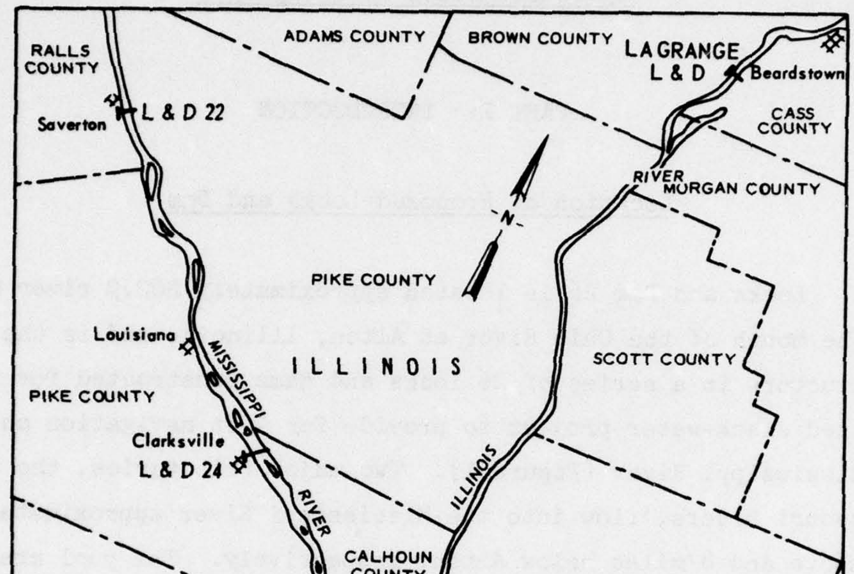

Figure 1. Vicinity map

occurred with a depth of only 5.5 ft over the lower sills of the locks. Since completion of the rock-fill dam at Chain of Rocks (mile 190.3) in 1963, the tailwater cannot fall below el 395.0.

History of Navigation Improvements on the Mississippi River

3. Prior to World War II, the navigation season in this region was considered to extend from 15 February to 15 December with very little traffic during winter months because of the danger to river craft caused by ice. Since that time, because of increased shipping demands and the use of more powerful steel hull towboats and heavier steel barges, year-round shipping has been maintained through the Mississippi River and the Illinois waterway access to Chicago except for short periods when the channel is blocked by ice.

4. Above Locks and Dam 26, the existing navigation project authorized by the River and Harbor Act of 30 August 1935 provides for a system of locks and dams to give a channel between the Missouri River and Minneapolis, Minnesota, 9 ft in depth and of adequate width for a long-haul commercial carrier service.

5. Below Locks and Dam 26 between the dam and the mouth of the Missouri River (8 miles), open-river regulating works, consisting of rock dikes and revetments, have been constructed to obtain project dimensions. Between 1932 and 1936, eight solid brush and stone dikes were added to those built in an earlier period to improve channel alignment and depth. As a result, there has been a decrease in the low-water elevation* throughout this reach, which reduced available depth over the lower miter gate sills of Locks 26. Under authority of the River and Harbor Act of 3 July 1958, a low-head rock dam constructed across the Mississippi River at Chain of Rocks eliminated this deficiency. The dam, which is nonnavigable, was essentially completed in September 1963. The entire authorized 9-ft navigation project has been in operation

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* The reference plane used as a basis for comparison is a flow of 30,000 cfs at Alton.

since 1940, with the exception of St. Anthony Falls extension, which was placed in operation in 1963. A new 1200-ft lock at Dam 19, Keokuk, Iowa, was opened to traffic in 1957.

6. In this area, the major products moved on the waterway, considering all tonnage passing through the locks, are about 29 percent petroleum and petroleum products; about 34 percent grain, mostly downbound; 16 percent coal; and 21 percent other commodities, such as sulphur, sand, steel, bulk chemicals, and other manufactured items.

7. The upper Mississippi River has been the subject of numerous reports by the Corps of Engineers, dating back to 1867, all of which have considered navigation improvements at one locality or another between the mouth of the Ohio River and Minneapolis, Minnesota. The principal documents which contain references to Locks and Dam 26 are as follows:

a. House Document No. 290, 71st Congress, 2nd Session. This document considered a 9-ft channel between the Illinois River and Minneapolis, Minnesota. This interim report was succeeded by House Document No. 137, 72nd Congress, 1st Session, which recommended extending the plan of improvement downstream from the mouth of the Illinois River (mile 218.0) to the mouth of the Missouri River (mile 195.0). It was similar to the interim report in that the project dimensions were to be obtained by a system of locks and dams supplemented by channel dredging when and where required.

b. House Document No. 136, 84th Congress, 1st Session. In this document, the District Engineer, U. S. Army Engineer District, St. Louis, recommended the construction of a commercial harbor 9 ft deep and about 2600 ft long, just below the downstream entrance to Locks 26. The same report also recommended the construction of a small-boat harbor 6 ft deep and about 800 ft long at the mouth of Piasa Creek, Illinois (mile 209.4).

Condition of Existing Locks and Dam

8. Founded on vertical wood and concrete friction piles, the structure, which consisted of a main lock 110 by 600 ft and an auxiliary lock 110 by 360 ft, plus 1724 ft of gated dam, was completed and opened for service in May 1938. In 1954, new upper lock gates were

installed in the main lock. Then a history of excessive deflection, settlement, underseepage, and loss of foundation material in the structure followed. Remedial measures have corrected the more serious and immediate deficiencies, but permanent repair is impractical because of engineering and cost considerations.

The Problem

9. The locks, located adjacent to Alton, Illinois, about 20 miles north of St. Louis, Missouri, not only are in poor condition structurally, but also are located such that the upstream approach channel is poorly aligned with the main lock because of a rock bluff jutting into the river about 1500 ft above the lock. Major repairs made to the locks over a period of years indicate the hard usage the locks have received and the difficulties attending navigation at this locality. The dam is also in need of some repair. Because of the increased shipping demands and the development of more powerful towboats and larger barge sizes, tow sizes have increased to lengths from 1100 to more than 1200 ft. This increase has resulted in a greater number of multiple lockages in relation to the total number of lockages made. Multiple lockages presently constitute a relatively high percentage of total lockages in this reach of the river. The existing locks exceeded their practical capacity (46,200,000 tons) in 1970 and traffic has been increasing for a number of years. In 1978 over 60 million tons passed through the locks. Present traffic congestion at the existing locks now results in delays averaging about 10-12 hours. As the traffic increases, congestion and attendant delays become longer and more common.

10. The magnitude of the change in tow size, character of traffic, and volume emphasizes the need for modern facilities to provide efficient service for present-day traffic as well as that which can reasonably be anticipated.

Description of Proposed Locks and Dams

11. The proposed locks and dam structure (see Preface for changes in authorized plan) will be located on the Mississippi River about 200.78 miles upstream from the confluence of the Mississippi and Ohio Rivers and about 2 miles downstream from the existing Locks and Dam 26. The dam and pool will be located in Missouri and Illinois with the locks on the Illinois side of the river. The two locks proposed will have dimensions of 110 by 1200 ft and will be separated by two 110-ft tainter gates to permit maximum utilization of the locks, with ported upper guard walls and solid lower guard and guide walls. The dam will consist of seven 110-ft-wide tainter gates and eight 150-ft piers, situated between the riverward lock on the left and an earth-fill overflow dike with crest el 422.0 on the right bank, and will provide a navigation pool about 40 miles long upstream to Lock and Dam 25. Normal upper pool elevation at the dam will range from 419.0 to 414.0 (same as the existing structure), and the minimum lower pool elevation will be 395.0 resulting in a maximum lift of 24 ft above pool 27.

Need for and Purpose of Model Study

12. The general design of Locks and Dam 26 was based on sound theoretical design practice and experience with similar structures; however, it was desired to ensure that the best arrangement and method of operation of the locks and dam were provided to eliminate any undesirable flow conditions that might make navigation conditions for tows entering or leaving the locks difficult or hazardous. Since navigation conditions vary with location and with flow conditions upstream and downstream of a structure, an analytical study to determine the hydraulic effects that may reasoably be expected to result from a particular design is both difficult and inconclusive. The location of the Locks and Dam 26 Replacement was not fixed at the time the model investigation was initiated. Therefore, a comprehensive model study was considered necessary for the following reasons:

a. To ascertain the best location, arrangement of the locks and lock walls, and composition and configuration of the dam, and the flow conditions that would result in the upper and lower lock approaches for various riverflows and lock and dam arrangements.

b. To assist in the development of any modifications that might be necessary to maintain two-way navigation in the area.

c. To eliminate any undesirable conditions.

d. To determine navigation conditions during construction of the project, and effects of the new structures on flood stage.

e. To demonstrate to navigation interests the conditions resulting from the proposed design and to satisfy these interests in regard to its acceptability from a navigation standpoint.

PART II: THE MODEL

Description

13. The model (Figures 2 and 3) reproduced a 6.7-mile reach of the Mississippi River, extending about 14,300 ft above and about 21,000 ft below the existing Locks and Dam 26. The model was of the fixed-bed

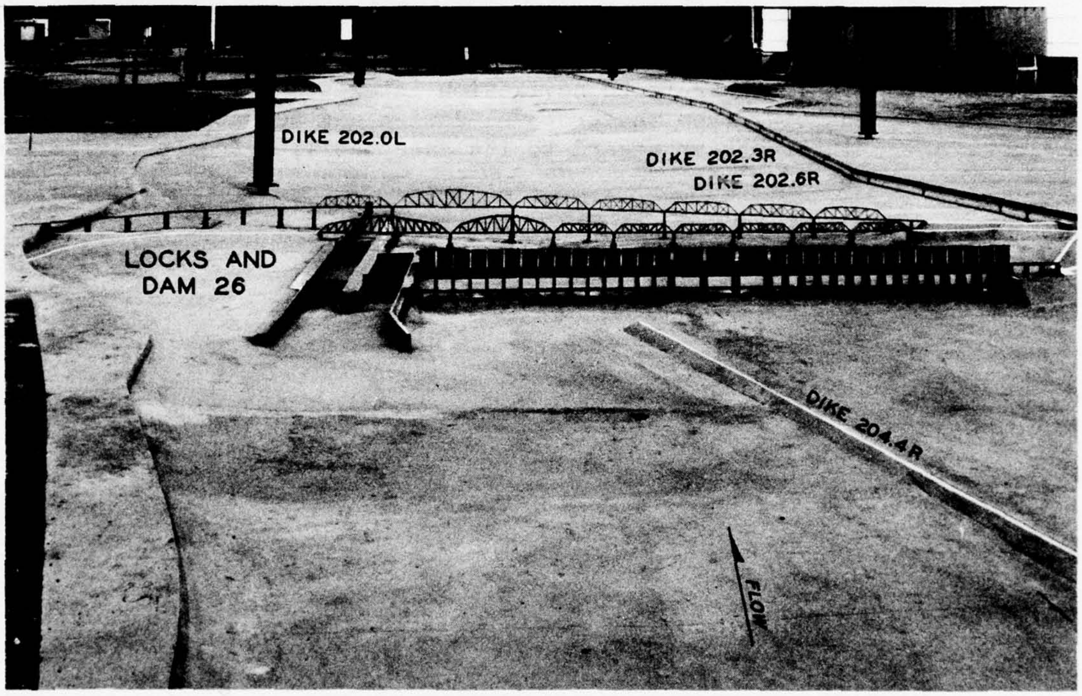


Figure 2. General view of model in the vicinity of the existing locks and dam

type, except for the channel in the areas of the existing locks and dam site and the proposed new locks and dam site in which pea rock was used to facilitate the installation of the proposed structures and modifications. The remaining channel and overbank areas were molded in sand cement mortar to sheet metal templates up to el 425.0. As the entire reach, except the upper portion, is contained within existing or proposed levees, this was considered sufficient for the reproduction and investigation of critical flows. The channel and overbank areas were molded to conform to a special topographic and hydrographic survey dated

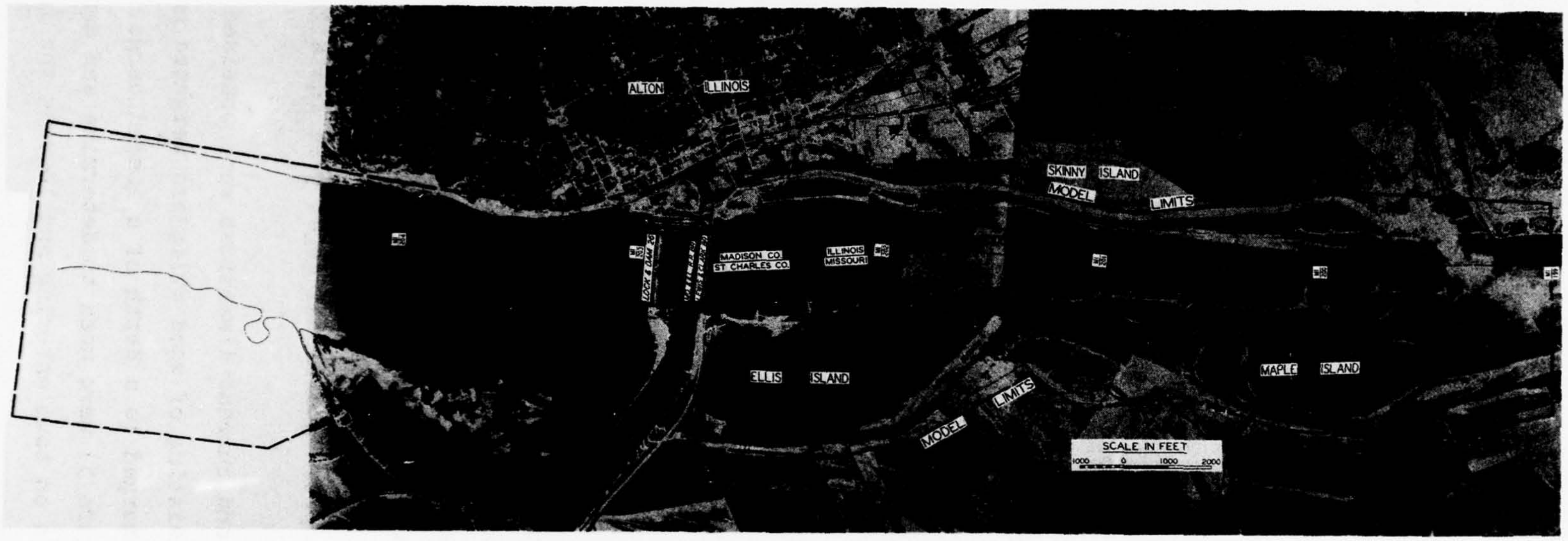


Figure 3. Aerial view showing existing prototype conditions and the area reproduced in the model

July 1966. The piers, locks, lock walls, dam, and bridges were constructed of sheet metal to preclude any change in elevation due to expansion or warping after the structures were set to grade. The lock and dam gates were simulated schematically with simple sheet metal slide gates.

Scale Relations

14. The model was built to an undistorted scale ratio, 1:120 model to prototype, to obtain accurate reproduction of velocities, cross-currents, and eddies that would affect navigation. Other scale ratios resulting from the linear scale ratio were as follows:

Area	1:14,400	Discharge	1:157,743
Velocity	1:10.95	Force	1:728,000
Time	1:10.95	Roughness (Manning's n)	1:2.22

Measurements of discharge, water-surface elevations, velocities, and force are transferable quantitatively from model to prototype equivalents by means of these scale relations.

Model Appurtenances

15. Water was supplied to the model by a 10-cfs centrifugal pump operating in a circulating system. Inflow was measured by means of two venturi meters of different sizes to permit control of the wide range in discharge. Water-surface elevations were measured by means of 20 piezometers located in the model channel (Figure 4) and connected to a centrally located gage pit. Additional gages were installed as required for special tests.

16. Velocities and current directions were obtained in the model by means of floats consisting of wood cylinders weighted on one end so that they would be submerged to a depth of a loaded barge. Two model tows and towboats (Figure 5) were used to determine and demonstrate the effects of currents on tows entering and leaving the locks. The

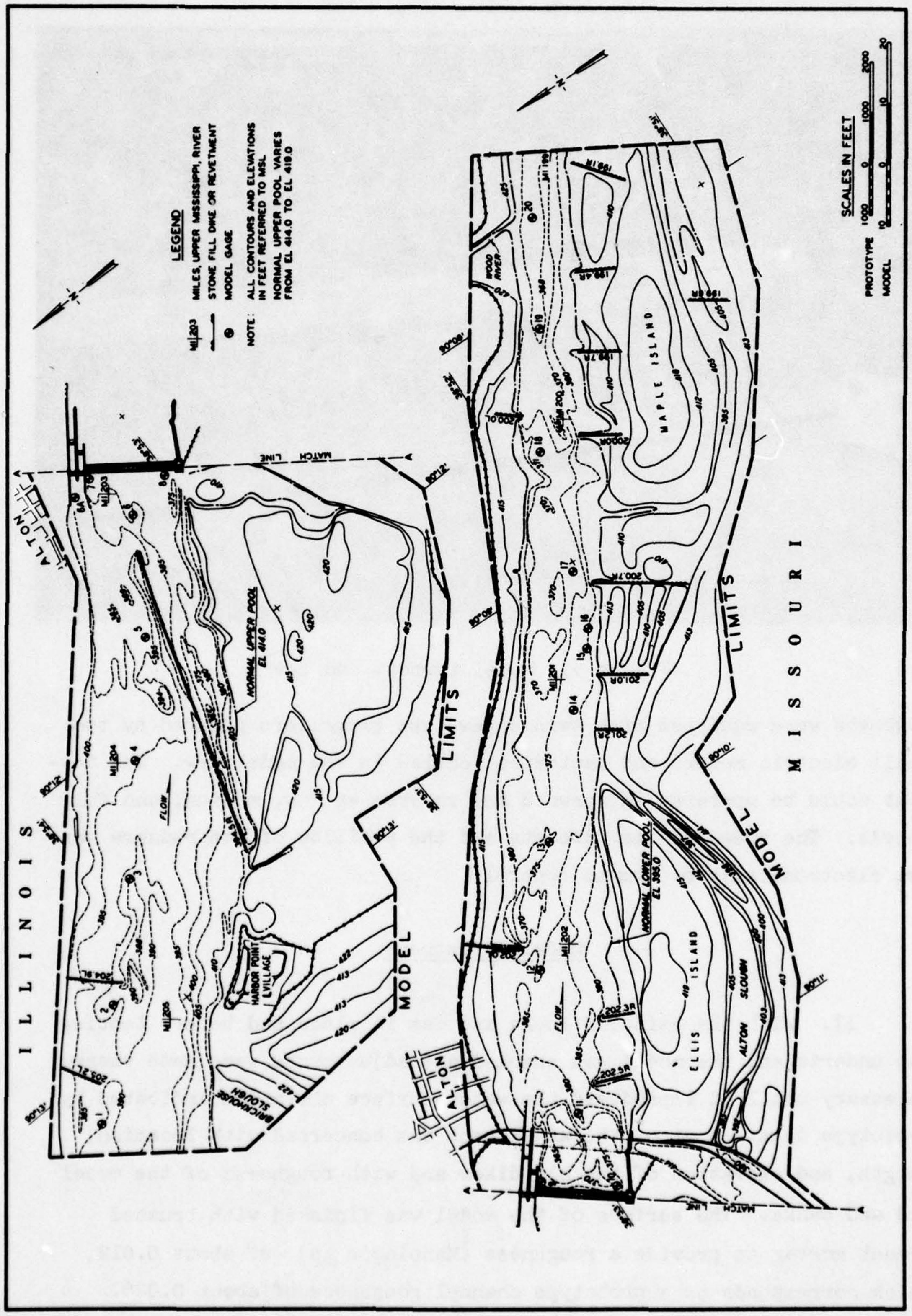


Figure 4. Model layout and location of gages

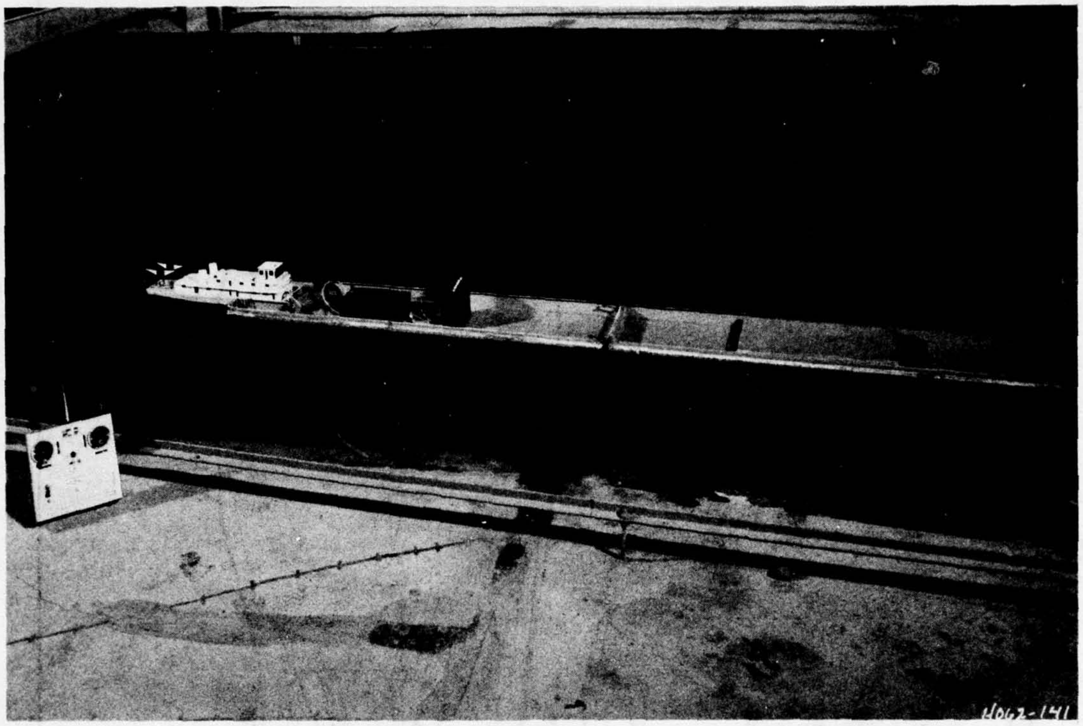


Figure 5. Model towboat and tow

towboats were equipped with twin screw-type propellers powered by two small electric motors and batteries located in the model tow. The towboat could be operated in forward and reverse at low, medium, and full speeds. The speed of the towboats and the position of the rudders were set electronically by remote control.

Model Adjustment

17. With the existing locks and dam in place and before testing was undertaken, the model was checked and adjustments were made where necessary until it reproduced the water-surface elevation indicated by prototype data. Most of the adjustment was concerned with location, length, and elevation of the old dikes and with roughness of the model bed and banks. The surface of the model was finished with brushed cement mortar to provide a roughness (Manning's n) of about 0.012, which corresponds to a prototype channel roughness of about 0.026.

Folded strips of 8-mesh screen wire were placed in areas covered by vegetation, as indicated by aerial photos, to obtain the higher roughness values along the islands and overbank. Tests conducted after adjustment of the model indicated that the model reproduced with reasonable accuracy the water-surface elevations furnished for this reach.

PART III: TESTS AND RESULTS

18. Tests were concerned primarily with modification of the existing structure and with a replacement structure for the development of two-way navigation. The results were based on a study of flow patterns and measurement of current velocities in the lock approaches and around the structures, swellhead at the dam, and the behavior of the model towboat and tow moving through the reach under various flow conditions. Tests were also conducted to determine the effects of the currents on navigation during the construction of the dam and the two locks, the tendency and location of scouring with the various cofferdam schemes, and modifications required to minimize any adverse effects. Several preliminary tests were made to develop modifications incorporated as part of specific plans. Little data were obtained during these tests and are not included in this report. Also, some of the data previously submitted to the St. Louis District are not included since they were used to develop a later plan.

Test Procedure

19. Tests of plans were conducted by reproducing representative stages and discharges. Flows in the Mississippi River selected for testing varied from 75,000 to 650,000 cfs. The maximum discharge at which normal upper pool elevation at the dam could be maintained and at which navigation through the locks would cease was dependent upon tailwater elevation below the dam, controlled to a large extent by the Missouri River discharge which entered the Mississippi River just downstream of the project. Normal upper pool was at el 419.0. Maximum navigable flow was controlled by a water-surface elevation of 430.0, which is 2 ft below the elevation of the top of the lock walls. The minimum regulated upper pool elevation is 414.0 based on drawdown conditions. The general plan of operation is to maintain normal pool el 419.0 at the dam. However, when the inflow from rainfall runoff causes the stage at Grafton, Ill., to rise to el 420.0, the pool at the dam is lowered to as low as

el 414.0 to maintain this elevation. The 5- and 10-yr frequency flood flows would be about 315,000 and 360,000 cfs, respectively. Controlled riverflows were obtained by introducing the required discharge and then manipulating the dam crest gates and the model tailgate until the desired upper pool and tailwater elevations were obtained. Uncontrolled riverflows were reproduced by using the required discharge and manipulating the tailgate to obtain the proper tailwater elevation for that flow. All flows were permitted to stabilize before any data were recorded.

20. Velocities were determined by timing the travel of floats described in paragraph 16 over a measured distance; current directions were ascertained by plotting the path of the floats with respect to ranges established for that purpose. In the plots of currents in turbulent areas or where eddies or crosscurrents existed, only the main trends are shown for clarity. Spot velocities were obtained with a calibrated miniature cup-type meter. General surface-current directions were determined by time-exposure photos recording the movement of paper confetti on the water surface. During tests with the model tows, the effects of currents on the movement of the towboats, drifting or powered, were observed.

Base Tests

Description

21. Base tests were conducted with conditions existing in the prototype before any modifications or improvements. The purpose of these tests was to provide information and data that could be used in determining the effects of the proposed modifications on water-surface elevation and current direction and velocities. The principal features of the existing structure, shown in Figures 6 and 7, included the following:

- a. Parallel locks were placed along the left bank adjacent to Alton, Illinois. The main lock, on the land side, was 110 ft wide by 600 ft long, and the auxiliary lock, 110 ft wide by 360 ft long.

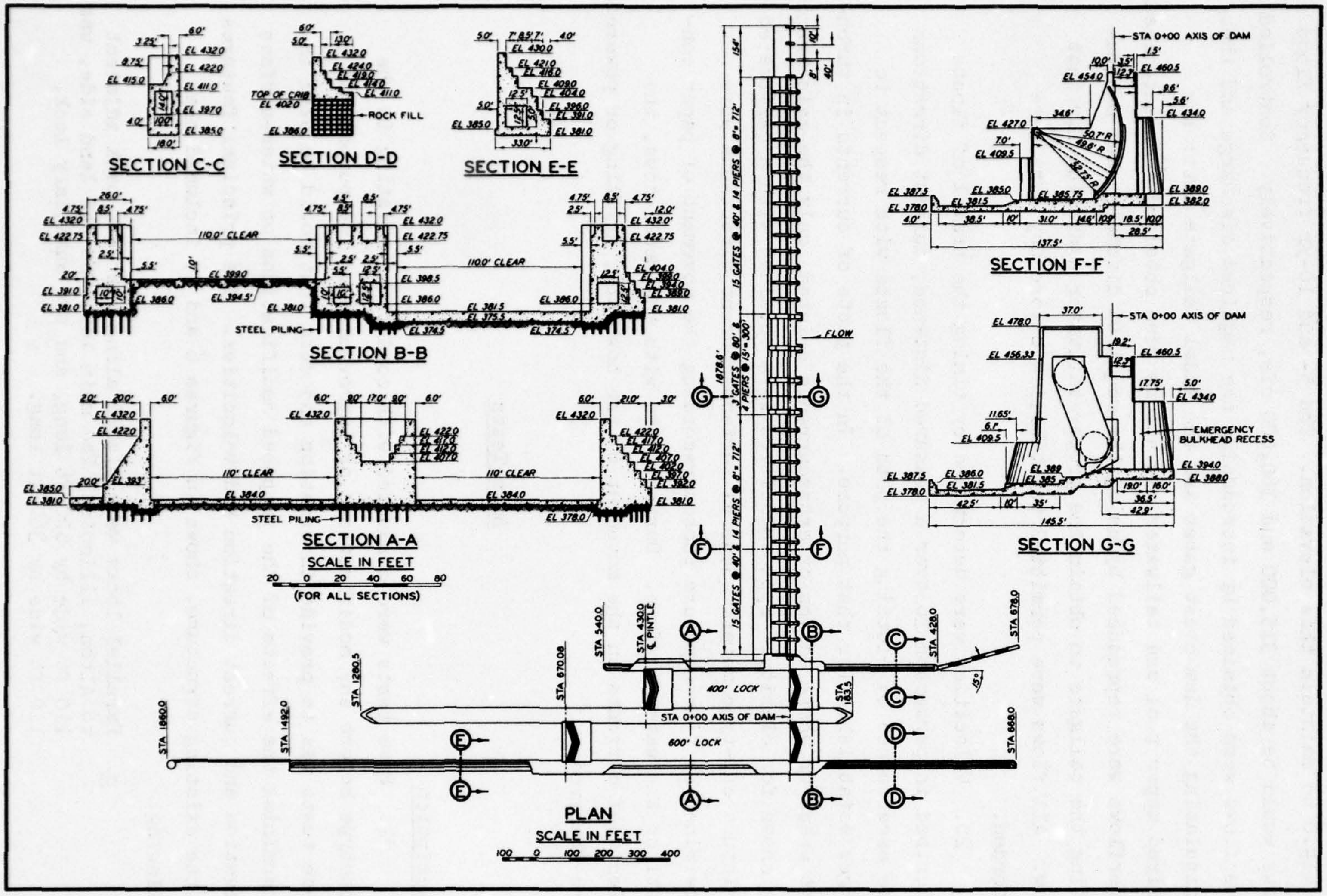


Figure 6. General plan and sections of existing structure

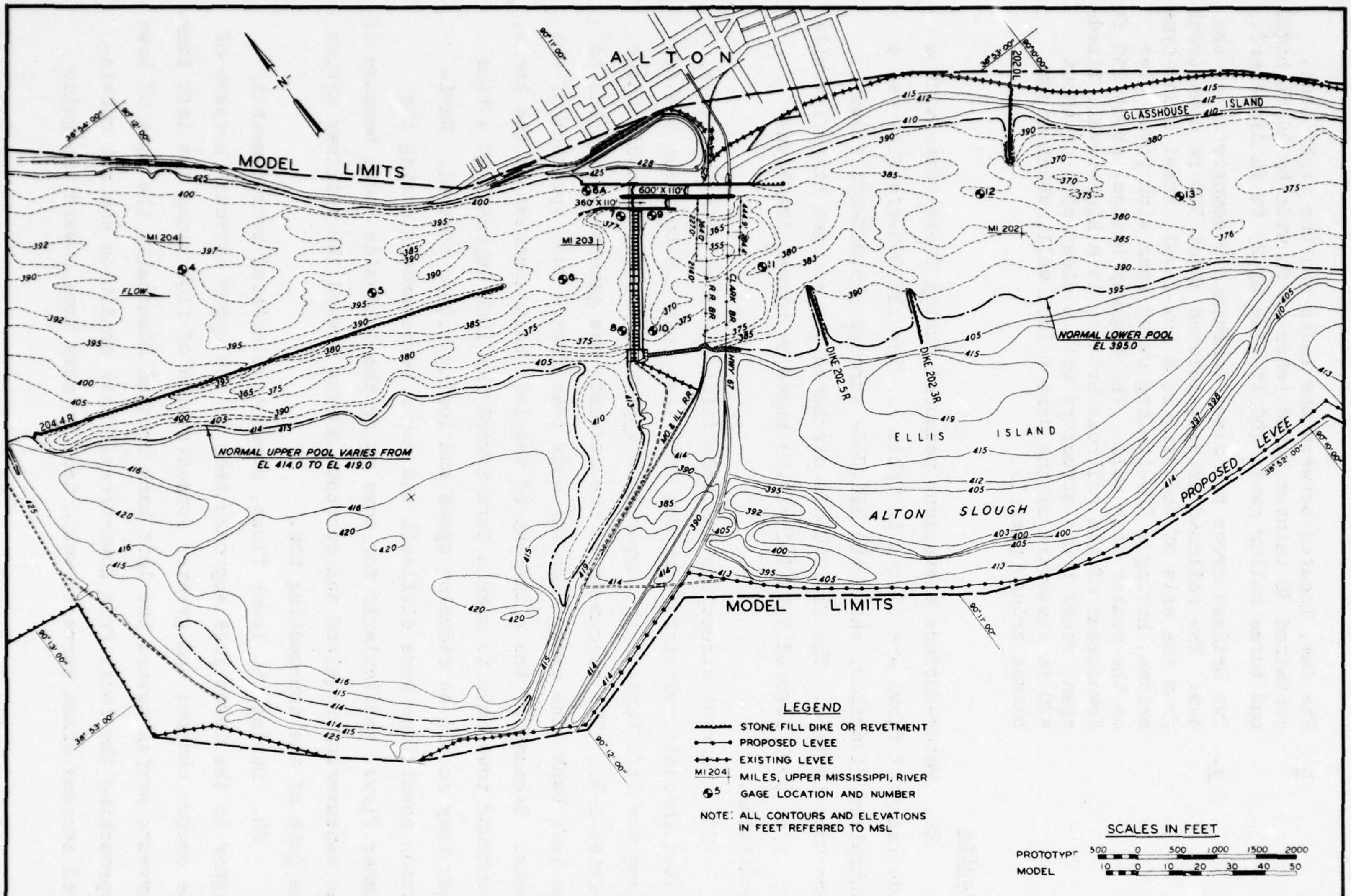


Figure 7. Vicinity map with existing conditions

b. The dam, located between the locks and the right bank, contained 30 tainter gates (each 40 ft wide by 30 ft high) and three roller gates (80 ft wide by 25 ft in diameter).

c. Two bridges cross the river immediately downstream of the dam. The railroad bridge, located about 920 ft downstream from the axis of the dam, is a low-level, steel truss-type bridge, having a 500-ft-long swing span with pivot pier on the center lock wall. The highway bridge, about 250 ft downstream of the railroad bridge, is a high-level, fixed-span, steel truss structure with a clear span of about 430 ft riverward of the center lock wall, which encompasses bridge pier 7.

Results

22. Water-surface elevations measured during these tests with a wide range of flows are listed in Table 1. The 117,000-cfs flow was a controlled riverflow, while the 312,000- through 545,000-cfs flows were uncontrolled. The 210,000-cfs discharge was an open riverflow with the upper pool lowered 5 ft (el 414.0) based on normal drawdown conditions.

23. Current directions and velocities obtained with the flows tested indicate velocities varying from less than 2.0 to about 5.8 fps along the left bank in the upper lock approach, depending on discharge (Plates 1-3). The alignment of the currents was generally parallel to the left bank line except for the eddy that formed just upstream of the locks. Because of the alignment of the left bank upstream of the locks, downbound tows have to make a turn toward the lock approach at a time when they should be reducing speed and losing rudder control. Navigation conditions were difficult and could be hazardous during the higher flows, particularly for tows using the land-side lock, because of the maneuvering required and currents moving toward the spillway across the path of the approaching tow.

24. During the lower flows, current velocities were somewhat higher in the lower lock approach than in the upper approach because of the deeper channel and greater concentration of flow along the left bank. Currents moving toward the left bank crossed diagonally the path of tows approaching the locks from downstream. This condition was not considered serious since currents would tend to move tows toward the guide

wall, and tows can maintain better control when traveling in an upstream direction.

25. The highest velocities were measured in the reach contracted by spur dikes along the right bank about 2.5 miles downstream of the existing structures. Velocities in this reach would be affected by the Missouri River discharge and changes in the bed during the higher flows.

Plan A

Description

26. Plan A involved the construction of two 1200-ft locks to replace the existing locks and the modification and rehabilitation of the existing dam. These locks would be separated to provide, insofar as practical, two-way traffic through the structure. Some of the features of this plan developed during preliminary tests were as follows (Figures 8 and 9):

a. The existing locks were removed except for the landward wall of the 600-ft lock and a section of the lower end of the intermediate wall encompassing the piers of the railroad and highway bridges.

b. Two locks, 110 by 1200 ft, were located within the existing dam riverward of the existing locks and offset by placing the riverward lock in the upper pool and landward lock in the lower pool. The locks were separated 140 ft between their outer walls. Seven 40-ft-wide gates were placed between the proposed landward lock and the remaining landward wall of the existing 600-ft lock. Three 40-ft-wide gates were installed between the two new locks with top of lock walls at el 434.5.

c. A 1200-ft-long upper guard wall was placed on the land side of each lock with top elevation same as lock walls. Fifteen 50-ft-wide ports were included in each guard wall with top of ports at el 385.0.

d. A 1200-ft-long lower guide wall was placed on each of the locks, one on the land side of the riverward lock and the other on the river side of the landward lock. Tops of guide walls were at el 430.0.

e. The two bridges were modified to provide for the horizontal and vertical clearances needed for approaching and passing through the locks.

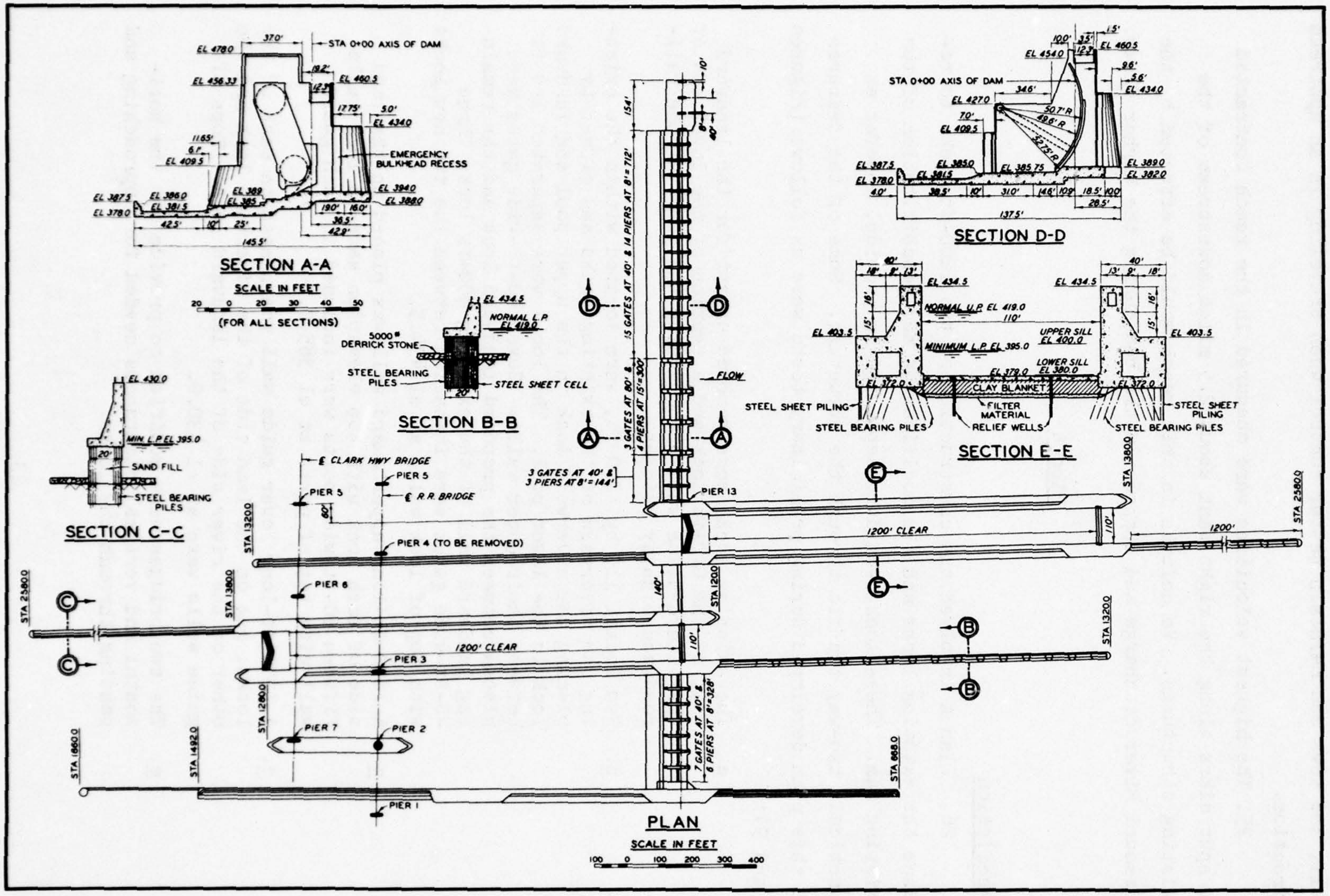


Figure 8. General plan and sections of plan A

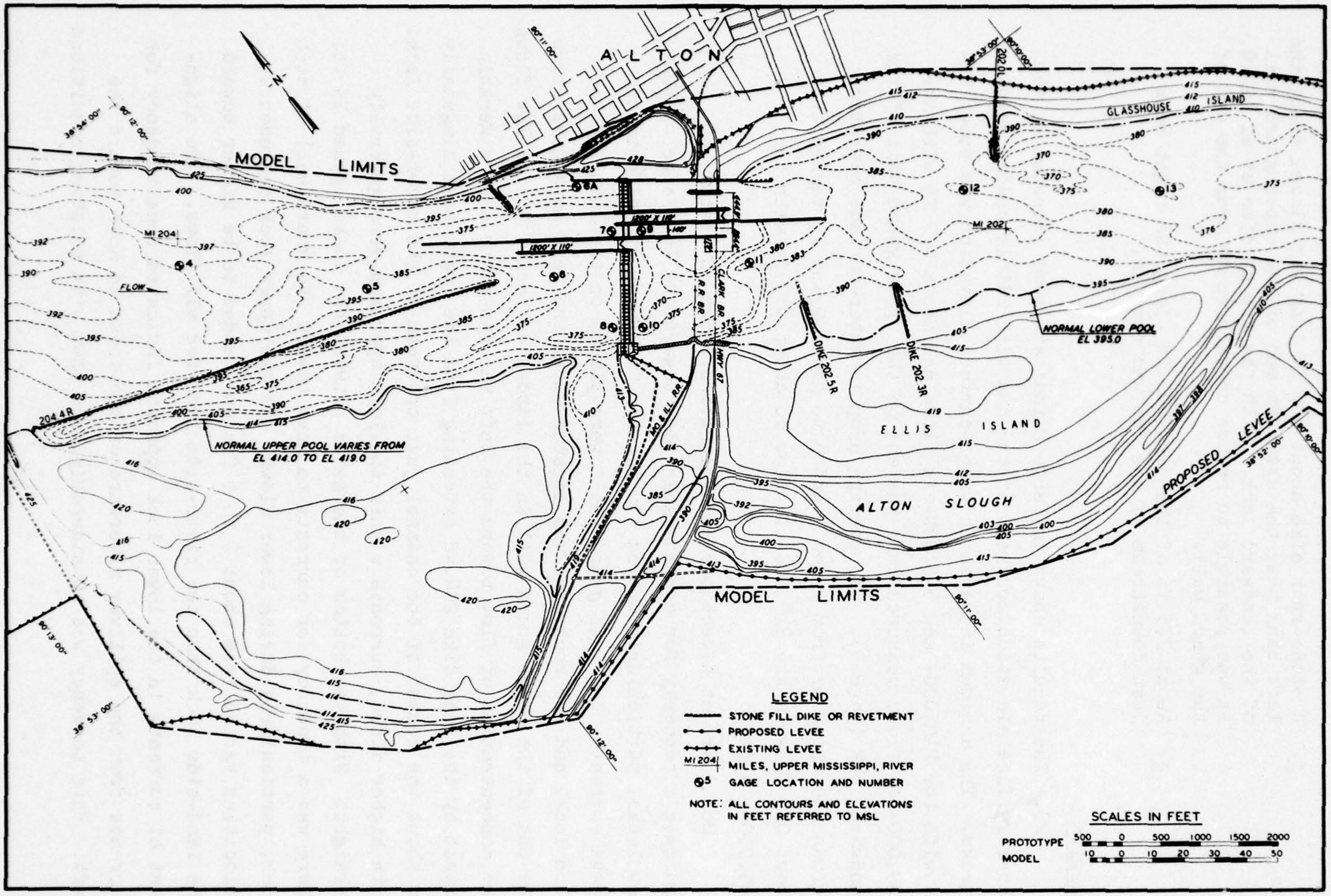


Figure 9. Plan A

f. A stone spur dike about 480 ft long was placed along the left bank near the upstream end of the upper guard wall of the landward lock. With the top of dike at el 421.0, it was possible to reduce crosscurrents near the end of the guard wall.

g. About 275 ft of the dike 202.5R along the right bank just downstream of the dam was removed.

Results

27. The results shown in Table 2 indicate some increase in the water-surface elevation upstream of the dam with little change downstream. The maximum increase in water-surface elevation was about 0.4 ft with the 210,000- and 545,000-cfs flows and about 0.7 ft with the 339,000-cfs flow, measured near the end of the upper guard wall of the landward lock (gage 5). The average drop in water-surface elevation across the spillway from gages 7 and 8 to gages 9 and 10 varied from about 0.2 ft with the 210,000-cfs flow to about 0.5 ft with the 545,000-cfs flow.

28. Current directions and velocities shown in Plates 4-6 indicate a tendency for considerable flow toward the left side of the new locks, particularly during low flows. To offset the effects of these currents, the lock guard walls were placed on the left side of each lock, and the rock dike was located along the left bank just above the end of the upper guard wall on the land-side lock. During low flows the concentration of flow toward the left was caused to a considerable extent by the long dike 204.4R extending from the right bank diagonally to about the center of the channel just upstream of the river-side lock. With higher flows, currents within the lock approach were generally straight. With the bottom of the gates between the locks opened 15 ft, there was a tendency for currents to move toward the guard wall of each lock, particularly during lower flows. In the upper lock approaches, velocities varied from about 2 to 6 fps and tended to be higher toward the land-side lock than the river-side lock for all flows. The alignment of currents in the lower lock approach indicated some tendency for currents from the spillway riverward of the locks to move toward the left. This tendency was considerably less than with existing conditions

(base test) because of the flow through the gates between the locks and those along the left bank landward of the locks. Velocities of currents in the lower approach to the land-side lock were considerably lower than in the approach to the river-side lock because of concentration of flow from the main spillway to the right of the locks and the shallow channel along the right side away from the locks. Some scouring of the bed, which can be expected in the reach downstream of the spillway and to the right of the lower guard wall of the land-side lock, should produce a reduction in velocities in the lock approach. Increasing the amount of opening of the gates between the locks would make it easier for downbound tows to approach the guard wall of the river-side lock but would tend to pull tows away from the guard wall of the land-side lock.

29. With the gates between the locks open a maximum of 15 ft and the top of the ports in the upper guard walls at el 385.0, no serious navigation difficulties were indicated in the approaches to either lock. Upbound tows would encounter higher velocities in the lower approach of the river-side lock and in the upper approach of the land-side lock.

Plans B and B-1

Description

30. Plan B was designed to replace the existing locks and dam with a new structure located about 2 miles downstream of the existing dam. The existing structures would continue to be used for navigation until the replacement structures are completed to the point that the upper pool can be maintained at the lower site. The existing riverward lock and the adjacent 11 gate bays would then be removed, and the navigation span at the railroad bridge would be modified to provide a clear span of about 420 ft riverward of the existing intermediate lock wall. The upper gate sills of the existing locks were lowered to el 400.0, and the lower guide wall of the landward lock was extended downstream about 700 ft with dumped rock to a top elevation of 432.0. The new structure, shown in Figures 10 and 11, had the following essential features:

a. Two parallel locks, 110 by 1200 ft long and 170 ft apart,

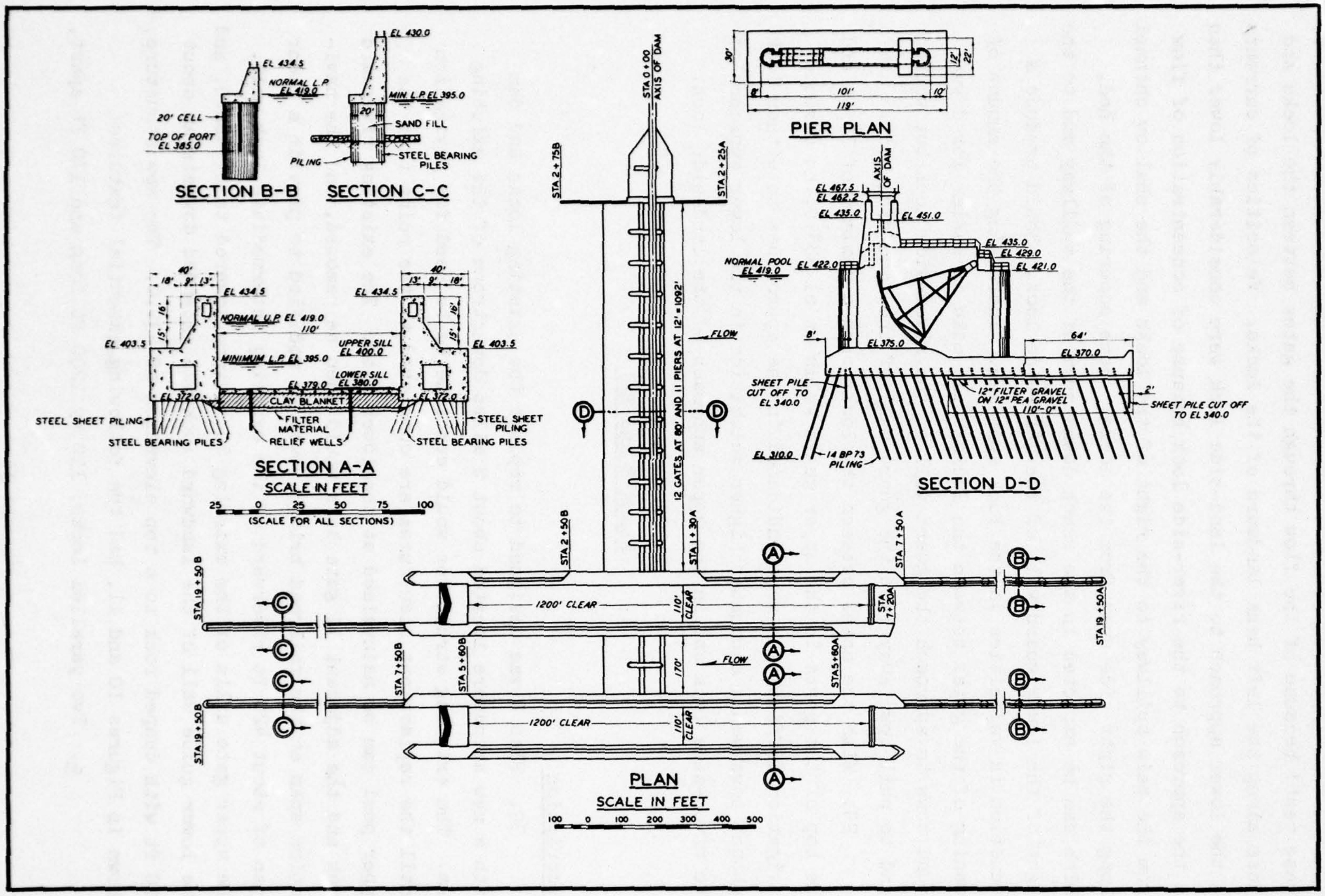


Figure 10. General plan and sections of replacement structures (plan B)

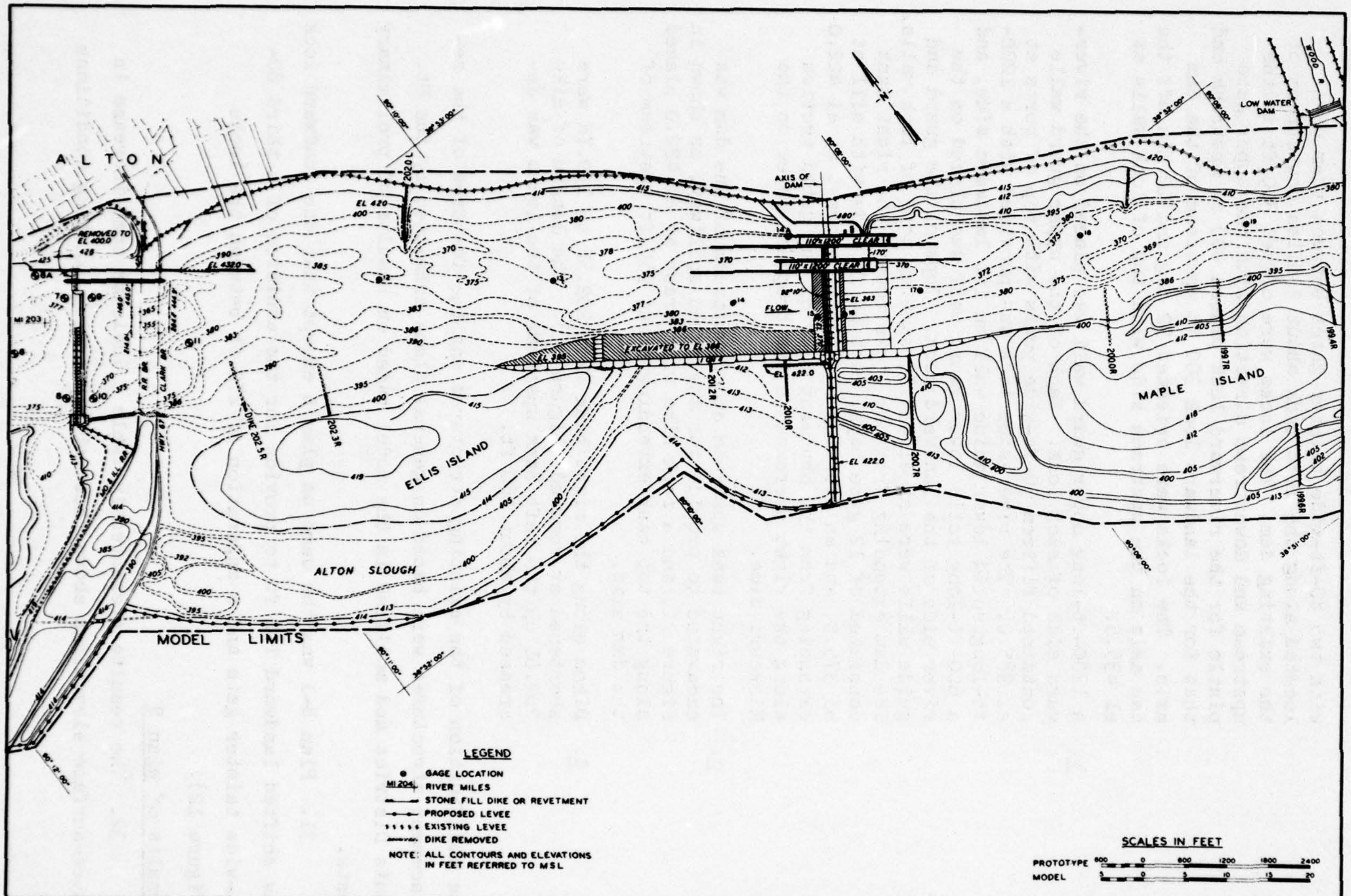


Figure 11. Plan B

with two 80-ft-wide tainter gates between them were located along the left bank about 2 miles downstream of the existing dam. The locks were offset 300 ft in the upstream and downstream direction with the upper gate pintle for the riverward lock placed 600 ft upstream and that for the landward lock 300 ft upstream of the dam axis. The locks were oriented 92 deg and 10 min off the dam axis on the upstream side with top of lock walls at el 435.5.

b. A 1200-ft-long upper guard wall was placed on the riverward side of each lock. Each of the upper guard walls contained fifteen 50-ft-wide ports with top of ports at el 385.0. The river-side lock was provided with a 1200-ft-long solid lower guide wall on its landward side, and a 600-ft-long solid lower guard wall was placed on the river side of the landward lock. Tops of the guard and guide walls were at el 434.5, same as tops of lock walls. The dam extending from the locks toward the right bank consisted of 12 gate bays, each 80 ft wide with sill at el 375.0, and an overflow section with crest at el 422.0 extending from the abutment pier of the gated section along the right overbank to the proposed levee on the Missouri side.

c. The right bank upstream and downstream of the dam was excavated to provide for the gated spillway, as shown in Figure 11, and a rock dike with crest at el 422.0 placed along the top bank extending about 950 ft upstream of the dam axis.

d. Dikes along the right bank from 202.5R to 199.7R were shortened or removed (Figure 11). The length of dike 202.0L on the left bank upstream of the locks was decreased by about 160 ft.

The modification of the existing structures and the features of the replacement structures were based on general plans submitted by the St. Louis District and modified in the model based on results of preliminary tests.

31. Plan B-1 was the same as plan B except that the landward lock was shifted landward 100 ft to provide for the addition of a third 80-ft-wide tainter gate and a separation of 270 ft between the locks (Figure 12).

Results of plan B

32. The results presented in Table 3 indicate some increase in water-surface elevations above those obtained with existing conditions

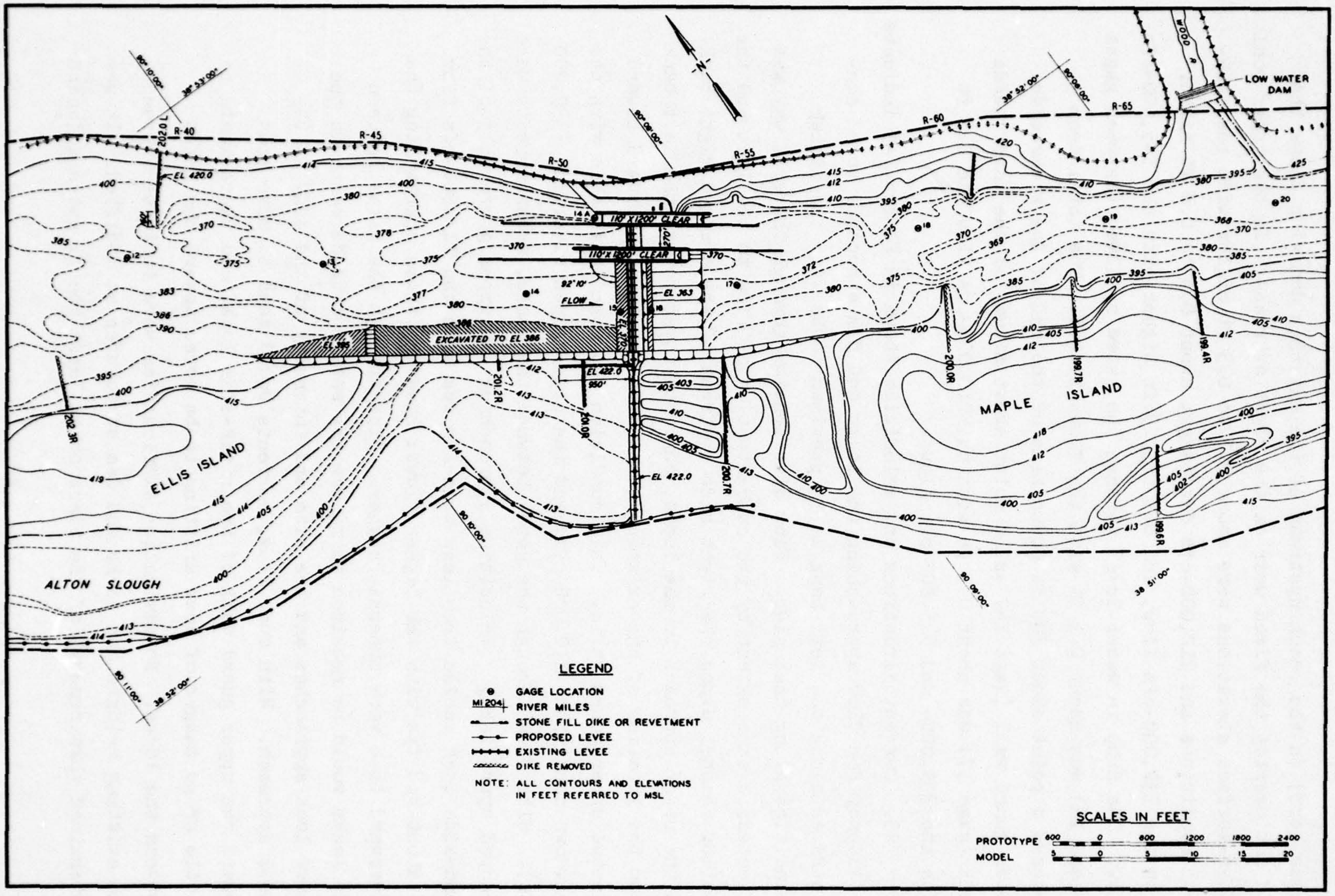


Figure 12. Plan B-1

(base test) in the reach upstream of the existing dam with flows that did not overtop the fixed weir on the right overbank. In the upper pool water-surface elevations were about 0.2 to 0.3 ft higher with the drawdown conditions and 210,000-cfs discharge, about 0.5 to 0.8 ft higher with the 339,000-cfs flow, and 0.3 to 0.5 ft higher with the 545,000-cfs flow. The drop in water level through the gated spillway (between gages 15 and 16) was about 0.1 ft with the flows tested. The total drop between a point about 300 ft downstream of the end of the river-side upper guard wall (gage 14) and a point about midway of the lower guide wall (gage 17) was about 0.3 ft with the 210,000-cfs flow and 0.5 ft with the 339,000- and 545,000-cfs flows.

33. Current directions and velocities shown in Plates 7-9 indicate a tendency for flow approaching the locks and dam structure to be concentrated along the left bank as in previous tests with the higher velocities along that side. Flow distribution through the spillway was affected to some extent by the concentration of flow to the left and the shallow channel along the right side upstream of the dredged right bank. In the upper approach to the locks, currents moved toward the left bank from the location of the existing structures and then riverward toward the two upper guard walls. This condition was more noticeable with the drawdown condition (210,000 cfs) and the higher open riverflow (339,000 cfs). With flow through the gates between the locks, crosscurrents were reduced appreciably. Velocities approaching the upper guard wall of the land-side lock varied from less than 2.0 fps with the 117,000-cfs flow to about 4.8 fps with the higher flows; however, those approaching the riverward lock were somewhat higher. Flow through the gates between the locks would be required to reduce the severe crosscurrents in the upper lock approaches and the size and intensity of the eddy in the lower approach. With such flow, currents would tend to move tows toward the upper guard wall of the river-side lock, and there would be little or no danger of tows drifting to be moved toward the gates between the locks. No navigation difficulties were indicated through the existing bridges and locks and dam structures as modified. It was determined that removal of the piers on at least the 11 gates adjacent

to the old locks would be required for best navigation conditions through the bridges. Conditions would also be better if the entire intermediate lock wall (riverward wall of existing 600-ft lock) was left in place. Furthermore, with the lock wall in place, pleasure craft and small tows could pass through the old lock chamber without interfering with traffic to the right in the main channel. The dike along the right bank, normal to the axis of the dam, would be required to improve flow through the gate bays near the bank during high flows which do not overtop the overflow section. Flow from the spillway moved at a slight angle toward the left and was affected to a considerable extent by the alignment and configuration of the channel downstream. Velocities along the center and to the right side of the channel were generally higher than along the left bank, but some adjustment of the channel configuration and a reduction in velocities can be expected with this plan. Currents in the lower approach to the river-side lock were toward and along the lower guide wall. Velocities in the approach downstream of the end of the lower guide wall of the river-side lock varied from about 5.8 fps with the lower flow to about 8.0 fps with the higher flow. Flow through the gates between the locks moved toward the left bank across the approach to the land-side lock. Velocities of currents moving toward the left bank were somewhat higher with the controlled riverflows than with open riverflow; the gates between the locks were opened the same amount as those in the spillway. Maximum velocity of currents between the locks varied from about 7.8 to 8.3 fps. There were no currents landward of the lower guard wall of the land-side lock. In the lower approach to the land-side lock, velocities varied from about 4.3 to 5.8 fps with the higher velocities occurring with the 117,000-cfs flow. However, velocities in the narrow channel about 4000 ft downstream of the guide wall of the riverward lock varied from about 6.0 to more than 8.0 fps.

34. No serious navigation difficulties were indicated in the approaches to either of the two locks. Downbound tows could be made to drift into the lock approaches from a considerable distance upstream after being properly aligned for the approach. In the lower approach to

the river-side lock, upbound tows moving riverward of the lower guide wall would tend to be moved toward the wall by currents from flow through the spillway.

Results of plan B-1

35. Water-surface elevations measured with this plan indicate a lowering upstream of the dam of about 0.1 to 0.2 ft with the 210,000- and 545,000-cfs flows and about 0.2 to 0.3 ft with the 339,000-cfs flow (Table 4). There was no change in the drop in water level through the spillway (gages 15 and 16), and the little change in water-surface elevations between gages 14 and 17 was about 0.1 ft less than in plan B. The differences in water-surface elevations between gages 14 and 17 were about 0.2 ft with the 210,000-cfs flow and about 0.4 ft with the 339,000- and 545,000-cfs flows. Downstream of the dam, water-surface elevations were not affected significantly.

36. The alignment of currents approaching the river-side lock in particular was affected to some extent by the increase in flow through the gates between the locks (Plate 10). Currents approaching the upper guard wall of the river-side lock were much straighter, and there was less tendency for crosscurrents near the end of the wall. Velocities of currents approaching the upper guard walls were not affected appreciably by the separation of the locks. The shift of the landward lock to the left reduced the amount of flow that the guard wall tends to intercept but did not have any significant effect on current alignment and velocities of currents approaching that lock. There was no appreciable difference in the currents in the lower lock approaches.

37. Navigation conditions for downbound tows approaching the river-side lock were generally better than with plan B, and tows could be made to drift toward the guard wall from a position further riverward because of the improvement in the alignment of the currents approaching the wall. Because of the increase in flow through the gates between the locks, there would be a greater tendency for tows to be moved toward the gates. However, this tendency did not appear to be serious since tows drifting with the currents would be moved toward the guard wall. Downbound tows approaching the land-side lock would encounter somewhat less

favorable navigation conditions than with plan B because of the reduction in the maneuver area available landward of the guard wall. However, no serious difficulties were indicated, and tows passing reasonably close to the end of the dike upstream could be made to drift into the lock approach. With the increase in the separation of the locks, there would be less tendency for tows moving in the same or opposite direction to interfere with each other.

Plan C

Description

38. Plan C was the same as plan B-1 except for the following modifications (Figure 13):

- a. The landward lock was placed in the lower pool with the upper gate pintle along the axis of the dam.
- b. The riverward lock was moved downstream with the upper gate pintle 400 ft upstream of the axis of the dam.
- c. Three cells were located about 100 ft apart, off the upper end of the landward wall of the river-side lock, angled about 20 deg landward of the alignment of the lock wall.
- d. A 20-ft-diam cell was located about 50 ft downstream and 50 ft riverward of the inner face of the riverward wall of the riverward lock.
- e. The upper approach to the landward lock was excavated to el 375.0.
- f. A section of dike about 700 ft long with top at el 420.0 was placed on the end of dike 202.0L and angled downstream generally parallel to the approach to the landward lock.

Results

39. Water-surface elevations listed in Table 5 indicate no significant difference between this plan and plan B-1. The increase in water-surface elevations with this plan was generally less than 0.1 ft except at gage 14A. This difference can be attributed to the location of the gage with respect to the structure.

40. Current directions and velocities approaching the river-side lock were about the same as with plan B-1 (Plates 11-13). In the

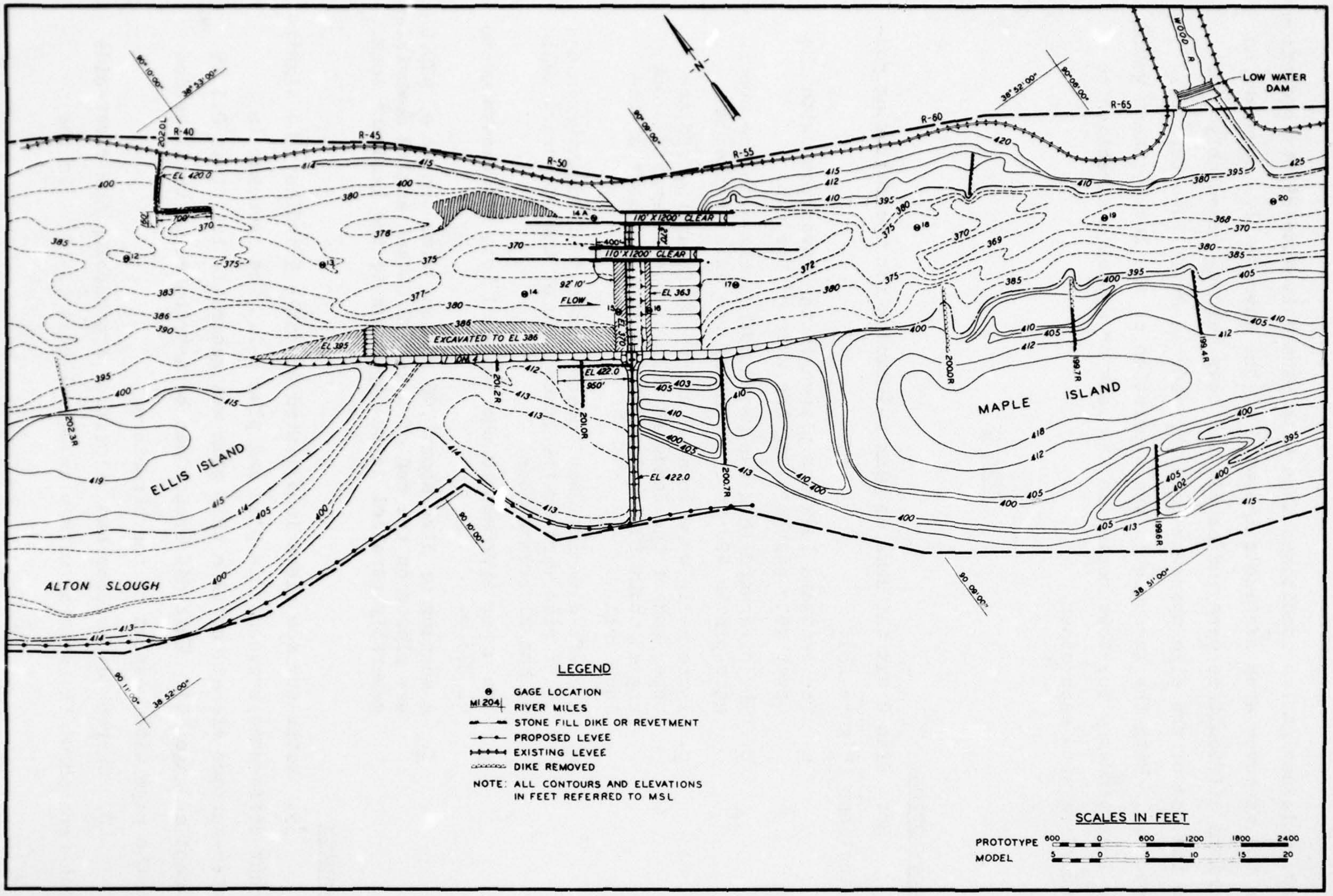


Figure 13. Plan C

approach to the land-side lock, there was a tendency for more of the flow along the left bank to move riverward before reaching the end of the upper guard wall, particularly with open riverflows. Since less of the flow moved landward of the guard wall, larger eddies formed between the wall and left bank. This condition was attributed in part to the alignment of the left bank and the position of the lock and guard wall with respect to that of the river-side lock. Currents and velocities in the lower lock approach were generally the same as with plan B but were affected locally by the shift of the locks toward the downstream. There was less current along the guide wall of the river-side lock with an eddy tending to form near the end of the river-side lock wall. The eddy was practically eliminated with a cell placed at the end of the wall. The alignment of currents in the lower approach to the land-side lock was generally better than with plan B, and velocities along the left bank in the approach were lower.

41. Navigation conditions in the upper approach to the river-side lock were about the same as with plan B-1, and no serious difficulties were indicated. Cells were placed at an angle from the end of the land-side wall of the river-side lock as a precautionary measure only since there appeared to be no serious danger of tows being moved toward the gates between the locks with proper operation of the gates. However, in the upper approach to the land-side lock, navigation conditions were not as good as with plan B-1, principally because of the reduced flow moving landward of the guard wall and the effect of the eddy between the wall and the left bank. Tows could be made to approach the wall with some power and rudder control without difficulty, but the head of the tow would tend to be moved away from the wall as it moved downstream of the end of the wall. Although not hazardous, additional maneuvering and the attachment of mooring lines to the wall would be required under some conditions for entrance to the land-side lock. With reduced velocities and better alignment of currents along the lower guide wall, navigation conditions in the lower approach to the river-side lock were better than with plan B-1. The cell at the end of the river-side lock wall would be required to offset the effect of the small eddy that would tend to move

the head of an upbound tow away from the guide wall as it approaches the end of the river-side lock wall. Navigation conditions in the lower approach to the land-side lock were affected by the limited maneuver area available between the lower guard wall and the left bank. Although conditions for upbound tows were somewhat better because of the lower velocities along the left bank, downbound tows would experience difficulties in moving the head of the tow riverward after passing the end of the wall since the stern could not be moved very far landward without being grounded.

Plan D

Description

42. This plan was the same as plan C except for the following (Figure 14):

a. The riverward lock, relocated normal to the axis of the dam, was 265 ft from and parallel to the landward lock. The emergency bulkhead recesses upstream of the upper gate pintles of both locks were placed on the axis of the dam. The dam consisted of eight 110-ft tainter gates with sills at el 375.0 and eight 15-ft-wide piers located between the river-side lock and right bank; and two 110-ft-wide tainter gates with three 15-ft-wide piers located in the separation between the locks. This arrangement of the locks and dam was then placed along the same alignment 350 ft downstream (mile 200.78).

b. The 1200-ft-long upper guard wall on each lock was ported with top of ports at el 404.0. The effective length of the riverward guard wall was extended upstream with two 20-ft-diam cells spaced 130 ft from the wall and 130 ft apart in line with the wall.

c. A 1200-ft-long solid lower guide wall was placed on the land side of the riverward lock, and a 900-ft lower guard wall on the river side of the landward lock.

d. The excavation of the right bank upstream and downstream of the dam was reduced by moving the landward limits of the excavation in line with the right abutment pier of the spillway. The channel between the right bank excavation and the river-side lock was dredged to el 370.0 upstream and downstream of the spillway.

e. The area between the locks downstream of the dam and the

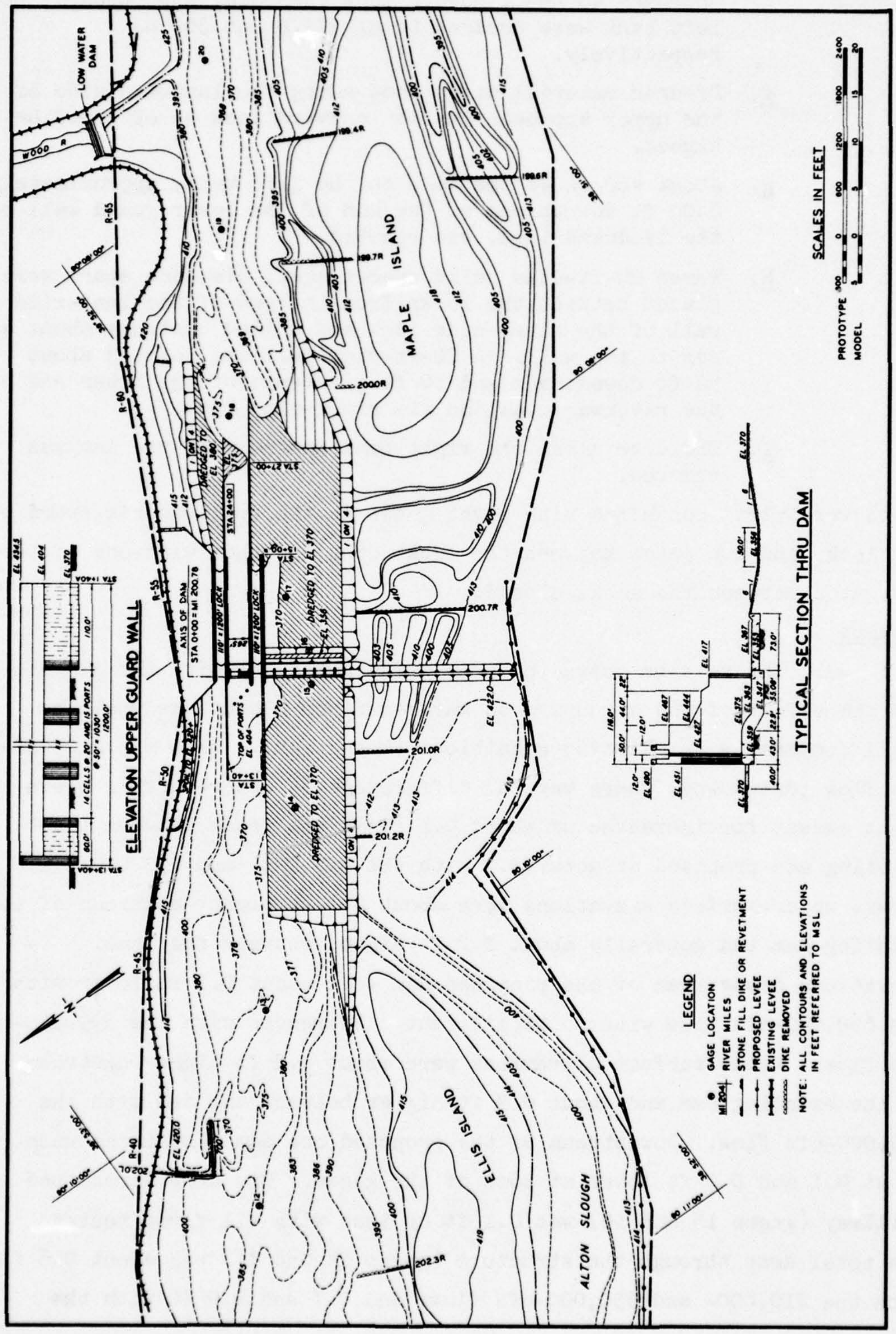


Figure 14. Plan D

approach to the landward lock including a portion of the left bank were dredged to el 375.0 and 380.0, respectively.

f. Dredged material was placed along the landward side of the upper approach to the landward lock to el 420.0 or higher.

g. About 400 ft of the dike on the left bank, approximately 2400 ft downstream of the end of the lower guard wall of the landward lock, was removed.

h. Three 20-ft-diam cells spaced equal distance apart were placed between the locks from the end of the land-side wall of the river-side lock and angled upstream about 45 deg to the wall. A 20-ft-diam cell was located about 50 ft downstream and 50 ft riverward of the lower end of the riverward wall on the river-side lock.

i. The dike along the right bank upstream of the dam was removed.

Tests were first conducted with eight gates in the spillway riverward of the locks and two gates between the locks open and then with one of the two gates between the locks closed.

Results

43. The results shown in Table 6 indicate that with the 10-gate dam the effect of the structure on water-surface elevations would be small compared with existing conditions (base test). With the 210,000-cfs flow (drawdown), there were no differences in water-surface elevations except for increases of about 0.1 ft in the reach between the existing and proposed structures. With the 339,000- and 650,000-cfs flows, water-surface elevations were about 0.1 ft higher upstream of the existing dam and generally about 0.2 ft higher between the dams. Elevations downstream of the proposed dam were about 0.1 ft lower with the 650,000-cfs flow with no significant differences with the 399,000-cfs flow. Water-surface elevations were about 0.2 ft higher upstream of the existing dam and about 0.3 ft higher between the dam with the 545,000-cfs flow. Downstream of the proposed new dam elevations were about 0.1 and 0.2 ft lower at some of the gages. The drop across the spillway (gages 15 and 16) was 0.1 ft or less with all flows tested. The total drop through the structure (gages 14 and 18) was about 0.6 ft with the 210,000- and 650,000-cfs flows and 0.7 and 0.9 ft with the

339,000- and 545,000-cfs flows, respectively.

44. The effects on water-surface elevations of closing one of the gates between the locks simulating the elimination of one gate were small, generally on the order of 0.1 to 0.2 ft higher upstream of the dam. There was practically no change in the drop across the spillway (gages 15 and 16), and the total drop through the structure (gages 14 and 18) was only about 0.1 ft higher than with both gates between the locks.

45. Since there were no navigation problems encountered with the two gates between the locks and the increase in losses with one gate was small, current directions and velocities were obtained only with the riverward gate between the locks closed. Little difference was indicated in the alignment of currents approaching the locks. The velocity of currents approaching the guard wall of the river-side lock showed some increase, and a somewhat larger eddy formed in the approach to the land-side lock (Plates 14-16). Because the closure of one gate reduced the flow between the locks, more of the total flow through the main spillway moved toward the left bank producing a greater angle in the alignment of currents across the lower approach to the land-side lock and causing a larger eddy to develop. Although there was some change in the alignment and velocity of currents in the lower approach to the river-side lock, the difference was not significant. Satisfactory navigation conditions were developed with either one or two gates between the locks. With one gate between the locks, conditions would be better with the gate in the center or close to the land-side lock. Conditions in the upper approach to the land-side lock were improved by filling along the left side of the approach channel to at least the elevation of the maximum navigable flow, since the filling would reduce the size and intensity of the eddy along that side and would provide a guide for tows moving close along the left bank.

46. There were no serious navigation difficulties indicated in the lower lock approaches with any of the flows tested. Placing a cell at the end of the river-side wall of the river-side lock eliminated the tendency for the small eddy to slowly move the heads of upbound tows away from the guide wall near the upper end of the walls. In the

approach to the land-side lock, the eddy currents would tend to move the heads of upbound tows away from the guard wall during some flows, but the effects would be small and could be overcome without difficulty. The alignment of currents in the lower approach to the land-side lock could be improved for navigation during the higher flows by filling along the left bank to an elevation above the stage for the maximum navigable flow.

Flow distribution

47. Flow measurements through the dam gates (Table 7) indicated reasonably good distribution for within-bank flows except for the gate in the main spillway next to the river-side lock, which is affected by flow through the ports of the river-side guard wall. With both gates between the locks open, each of the gates would pass less of the total flow than most of the gates in the main spillway. With only one gate between the locks open, the flow through that gate would be higher than the average of the gates in the main spillway. Flow through the gate bays near the right abutment would be affected by the configuration of the channel upstream and the concentration of flow along the left side. With overbank flows that do not overtop the overflow section, currents moving parallel to the axis of the dam from the overbank toward the spillway created a disturbance in front of the two gate bays near the right bank reducing discharge through those gates. This condition could be minimized with the dike along right bank as in plan C.

Velocity measurements

48. Measurements of velocities near the channel bed were made to provide some indications of the protection that might be required with one gate in the spillway fully open and with one gate half open and a head on the dam of 24 ft, based on emergency conditions and conditions that might be necessary to pass ice and debris. The results presented in Tables 8 and 9 indicate velocities measuring more than 25 fps near the end sill and decreasing gradually toward the downstream. Velocities near the end sill tended to be slightly higher with a gate half open than with the gate fully open. Although the bed material in the model consisted of coarse pea gravel of 1/4- to 3/8-in. grain size, which was not

to scale, the maximum depth of scour varied from 20 to 32 ft and the scour area extended from 200 to 360 ft downstream of the end of the stilling basin. Somewhat deeper scour holes developed downstream of the end of each of the gate piers adjacent to the open gate. Scouring of the bed without protection could extend a considerable distance farther downstream than indicated by scouring in the model, as shown in Tables 8 and 9.

Plan D Cofferdam Tests

Model preparation

49. In order to determine the best construction sequence for the replacement structure, navigation conditions during the construction, and the degree of protection required for the various cofferdam stages, completed structures, and adjacent banks, special movable-bed tests were conducted. For these tests a portion of the model in the reach that could be affected by the structures was converted to provide for a movable bed and adjusted to reproduce the movement of sediment that could reasonably be expected in the reach under various flow conditions. Since one of the cofferdam plans proposed the diversion of a portion of the riverflow and navigation through an excavated bypass canal along the right overbank, this canal with a movable bed was included in the model.

50. The adjustment of the model involved modifying the slope and discharge scales and determining the rate of feed of bed material and a time scale that appeared to be reasonable. Operation of the model during the adjustment and some of the testing was based on a typical hydrograph dated 1962. The final adjustment had to be based on judgment and experience with movable-bed models since sufficient data normally required for adjustment and verification were not available.

Test procedure

51. The testing was based on the assumption that the bed in the prototype consisted of unconsolidated sand to the maximum depth of scour and that all flows were confined by existing and proposed levees. Before the start of each new test, the bed of the model was remolded to

conform with the hydrographic survey dated July 1966. The left bank line adjacent to the location of the proposed new locks was excavated to the ultimate grade and maintained throughout the tests. The flows and tailwater elevations reproduced during nearly all of the scour tests were the most severe conditions that could be expected and were based on the following flows and sequence:

	Discharge, cfs	Tailwater El
(1) Maximum drawdown (average Missouri River flow)	210,000	413.0
(2) 5-yr flood	315,000	425.2
(3) Maximum drawdown (low Missouri River flow)	240,000	412.8
(4) Maximum drawdown (average Missouri River flow)	210,000	413.0
(5) 10-yr flood	360,000	427.8
(6) Maximum drawdown (low Missouri River flow)	240,000	412.8

Before the start of each test, water-surface elevations and measurements of current directions and velocities were taken using the theoretical velocity and discharge scale without any scouring of the model bed. The study of navigation conditions with the model towboat and tow was accomplished before and sometime after the movable-bed tests but with the correct discharge and velocity scales rather than with the scales needed to develop scour patterns. The bed of the model was surveyed at the end of each test.

Base test

52. A base test was conducted simulating conditions existing in the prototype from information available before the start of construction. The purpose of this test was to determine current directions and velocities and water-surface elevations with the flows selected for testing, which would be used as a basis for comparing the effects of various cofferdam schemes. Tailwater elevations were controlled just downstream of the site for the proposed structure (gage 17). The water-surface elevations (Table 10) would be used to determine the effects of the various cofferdam plans on stages and slopes. Current directions and velocities shown in Plate 17 indicate currents through the reach to be generally straight with velocities of less than 5.0 fps with the

5- and 10-yr frequency flows and slightly higher with 210,000-cfs drawdown conditions. Along the right overbank near the axis of the proposed dam, velocities varied from about 1.4 to 1.9 fps with the 5-yr frequency flood (315,000 cfs) and from 1.7 to 2.5 fps with the 10-yr frequency flood (360,000 cfs).

Tests of plans

53. Preliminary tests were conducted of many plans and sequences for construction involving different arrangements and stages of construction. These plans included both two-stage and three-stage cofferdams with and without a diversion canal on the right overbank and with the access road extending from the right bank levee to the structures near the right bank at elevations of 414.0 and 422.0. The sequence of construction was also varied based on starting construction on one side of the river or the other and maintenance of navigation during construction. Use of a bypass canal along the right overbank for navigation and diversion of some of the river flow did not prove to be satisfactory during the preliminary tests and was eliminated from consideration. Results indicated that downbound tows attempting to use the bypass canal would experience considerable difficulties in entering the canal during all flows after having to maneuver within a short distance to cross from one side of the channel to the other side when leaving the existing locks along the left bank. Tows would also encounter difficulties in overcoming the effects of currents moving across the canal during flood flows. Because of the location of the entrance and the alignment of the bypass canal with respect to the alignment of currents, shoaling and deterioration could be expected near the entrance to and in the lower reach of the canal.

54. The various cofferdam stages tested indicated that severe scouring would occur near the upstream riverward corner of each cofferdam and extend along its riverward face. Furthermore, it was determined during these tests that the scouring could be moved away from the main cofferdam by placing a deflector consisting of an upstream extension of the riverward arm of the cofferdam with a section on the upstream end angled about 45 deg away from the direction of flow.

First-stage cofferdam

55. Description. The final plan considered to be the most feasible on the basis of its effect on channel deterioration, water-surface elevations, and navigation involved construction in three stages. The first stage would require a cofferdam extending from the Missouri side riverward to provide for the construction of the six and one-half gate bays on the right side of the eight-gate spillway. This provided a navigation pass about 690 ft wide between the cofferdam and the left bank (Plates 18 and 19). This stage was later modified based on the use of a seven-gate spillway. On the upper corner of the riverward side of the cofferdam, a deflector was placed consisting of an upstream extension of the riverward face about 160 ft and then a section about 193.6 ft long angled 45 deg landward (Photo 1a). The deflector was designed for construction with sheet pile cells of the same size as those used in the main cofferdam since the material could be removed and reused. The rock protection for the deflector was 10 ft high with a 10-ft-wide crown extending from the upstream end of the angled section to just downstream of its junction with the main cofferdam.

56. Results. Water-surface elevations shown in Table 11 indicated that the water surface just upstream of the cofferdam (gage 14) would be about 0.8 ft higher with the lower flow to about 0.7 ft higher with the flood flows than with existing conditions (base test). With the higher flows, water-surface elevations at the cofferdam were about 426.2 for the 5-yr flood and about 429.0 for the 10-yr flood. The total drop between gages 14 and 17 (gage 14 located 1720 ft upstream of the axis of the dam and gage 17 about 990 ft downstream of the axis) ranged from about 1.5 ft with the 240,000-cfs flow to 0.8 ft with the 360,000-cfs flow. Water-surface elevations at these gages were affected by their location with respect to the cofferdam and would tend to indicate a greater drop than if the the gages were located along the center line of the navigation pass. Scouring of the bed between the cofferdam and the left bank, which can be expected, would tend to lower water-surface elevations at the upper gage and thus reduce the drop through the contracted reach. The scour pattern developed in the model reproducing

drawdown and flood flows only indicates that the maximum scour occurred about 150 ft from the deflector to a depth of about el 323.0 (Plate 18). Maximum scour along the main part of the cofferdam occurred about 75 ft from the structure down to about el 340.0 with little scouring adjacent to the cofferdam. Current directions and velocities shown in Plate 18 indicate currents along the left bank to be generally straight and parallel to the bank with maximum velocities of about 6.0 to 6.7 fps with the flows tested. Spot velocities were from 1.0 to 3.0 fps higher through the pass (Plate 19). The highest bottom velocities were measured near the upper end of the deflector and ranged up to about 9.5 fps. Bottom velocities along the riverward side of the main cofferdam were lower along the upper end and increased to a maximum of about 4.3 fps near the lower end. The highest velocities were measured with the 240,000-cfs flow under drawdown conditions. The current directions and velocities were obtained with the bed of the model between the cofferdam and the left bank maintained at el 370.0 before the scour pattern had developed.

57. No serious navigation difficulties were indicated for upbound or downbound tows through the pass between the cofferdam and the left bank. Upbound tows would need sufficient power to overcome current velocities on the order of at least 7.0 to 8.0 fps. Downbound tows could be made to drift through the pass from a considerable distance upstream with little danger of hitting the cofferdam.

Second-stage cofferdam

58. Description. The second-stage cofferdam was located near the center of the channel and provided for the construction of the remaining one and one-half gate bays on the 8-gate spillway and the riverward lock. With this stage cofferdam, a navigation pass about 375 ft wide would be provided along the left bank, and five of the completed gate bays would be open for the passage of a portion of the riverflow (Plates 20 and 21, Photo 1b). A deflector was provided on the upstream end of the left face of the cofferdam similar to that provided for the first-stage cofferdam. To reduce scour near the upper right corner near the completed portion of the spillway, a curved rock dike was placed along that

portion of the cofferdam from the spillway and flared to provide a guide wall for flows moving toward the left side of the spillway. The dike had a top elevation of 380.0 and a crown width of 10 ft.

59. Results. Water-surface elevations shown in Table 12 indicate that river stages just upstream of the cofferdam (gage 14) would be about 0.5 ft to 0.3 ft higher than with existing conditions (base test) with the 5- and 10-yr flood flows, respectively. The elevations of 426.0 for the 5-yr flood and 428.6 for the 10-yr flood were about 0.2 to 0.4 ft lower than those obtained with the first-stage cofferdam. The total drop between gages 14 and 18 (gage 18 located about 3,350 ft downstream of the dam axis) ranged from about 0.8 ft with the 240,000-cfs flow to about 0.4 ft with the 360,000-cfs flow.

60. The scour pattern developed during the test of the second-stage cofferdam (Plate 20) indicates scouring upstream of the spillway near the cofferdam, downstream of the spillway near the right bank, and in the navigation pass off the corner of the deflector. The scouring downstream of the spillway and upstream along the rock dike reached el 324.0 and 341.0, respectively. Scouring off the end of the cofferdam deflector reached el 333.0 about 75 ft from the deflector. There was some relatively small scouring along the left bank. Except for the upper portion of the left face of the cofferdam, there was no indication of serious scouring along the navigable pass side of the cofferdam. Some shoaling occurred in the lower approach to the navigable pass resulting mostly from material scoured upstream. Current directions and velocities affecting navigation shown in Plate 20 indicate currents along the left bank and through the pass between the cofferdam and the left bank to be reasonably straight and generally parallel to the sailing line. Maximum velocities between the cofferdam and left bank were on the order of 6.0 to 6.7 fps and about the same as with the first-stage cofferdam. No serious navigation difficulties were indicated for upbound or downbound tows. Downbound tows moving close to the cofferdam would have to maintain some rudder control and a clearance of at least 50 ft between the cofferdam and tow. Because of the limited width of the pass, one-way traffic might be desirable during some flows.

61. Spot velocities measured during tests of this stage cofferdam (Plate 21) indicate the highest velocities to be near the bend in the cofferdam deflector (8.2 fps) and along the rock dike at the upper right corner of the cofferdam (8.0 fps) with 240,000 cfs under drawdown conditions. Along the face of the cofferdam, maximum velocities were generally moderate, on the order of 4.0 to 4.7 fps along the navigable pass, and about 2.9 fps along the spillway side downstream of the dam. However, maximum velocities along the toe of the left bank slope on the Illinois side were about 4.2 fps and along the right bank on the Missouri side about 4.3 fps.

Third-stage cofferdam

62. Description. The third-stage cofferdam was located on the Illinois (left) bank and provided for the construction of the landward lock and the two gate bays between the locks. With this stage cofferdam in place, the gated spillway and the riverward lock would be completed and in operation (Photo 1c). Traffic moving through this reach of river would have to use the completed lock during construction. The upper pool could be controlled at the new structure when the closure between the lock and left bank is completed.

63. Results. Water-surface elevations with this cofferdam stage (Table 13) were about 0.8 ft and 0.5 ft higher upstream of the dam (gage 14) with the 5- and 10-yr flood flows, respectively, than with existing conditions (base test). As compared to the first-stage cofferdam, el 426.3 at gage 14 for the 5-yr flood was 0.1 ft higher, and el 428.8 for the 10-yr flood was about 0.2 ft lower. The total drop between gages 14 and 18 ranged from about 1.0 ft with the 240,000-cfs flow to about 0.6 ft with the 360,000-cfs flow.

64. The scour pattern developed during this test indicates considerable scouring just upstream of the spillway along the right side of the upper guard wall and just downstream of the spillway along the wall of the river-side lock near the Missouri bank (Plate 22). Maximum scour along the lock wall and upper guard wall reached el 335.0 and along the Missouri bank just downstream of the next to last gate pier to el 324.0.

65. Current directions and velocities shown in Plate 22 indicate

currents along the left bank in the upper lock approach to be moving toward the upper guard wall with little crosscurrent near the end of the wall. A counterclockwise eddy formed along the upper arm of the cofferdam but did not extend far enough into the approach to affect currents moving toward the guard wall. Maximum velocities in the upper approach to the lock were on the order of 4.0 to 4.5 fps and lower closer to the guard wall. Currents in the lower approach moved at a small angle from the spillway toward the left bank. Maximum velocities along the lower guide wall of the completed lock were about 5.0 to 5.3 fps and slightly higher away from the wall. No serious navigation difficulties were indicated for upbound tows approaching or leaving the lock. Spot velocities (Plate 23) indicate high bottom velocities along the lower end of the upper guard wall of 4.8 fps with the 240,000- and 360,000-cfs flows. Maximum velocities along the lock just downstream of the spillway were as much as 5.5 fps and about 4.0 to 5.3 fps farther downstream with the 240,000-cfs flow under drawdown conditions. Velocities along the toe of the right bank were as high as 4.5 fps upstream and somewhat lower downstream. The scour pattern and velocity measurements indicate that some protection of the bed would be required along the lower third of the upper guard wall to prevent scour of the bed which would increase flow through the ports and affect currents in the upper lock approach.

Plan E

Description

66. Plan E was the same as plan D except for the following (Figure 15):

- a. The number of spillway gates was reduced to seven. Excavation along the Missouri bank was reduced based on the revised length of gated spillway. The excavation of the right bank and channel along the right side to el 370.0 was reduced to about 2150 ft upstream and 1450 ft downstream of the dam, and a rock dike with top el 422.0 was placed along the top bank extending from the overflow section to about 600 ft upstream. The right bank and

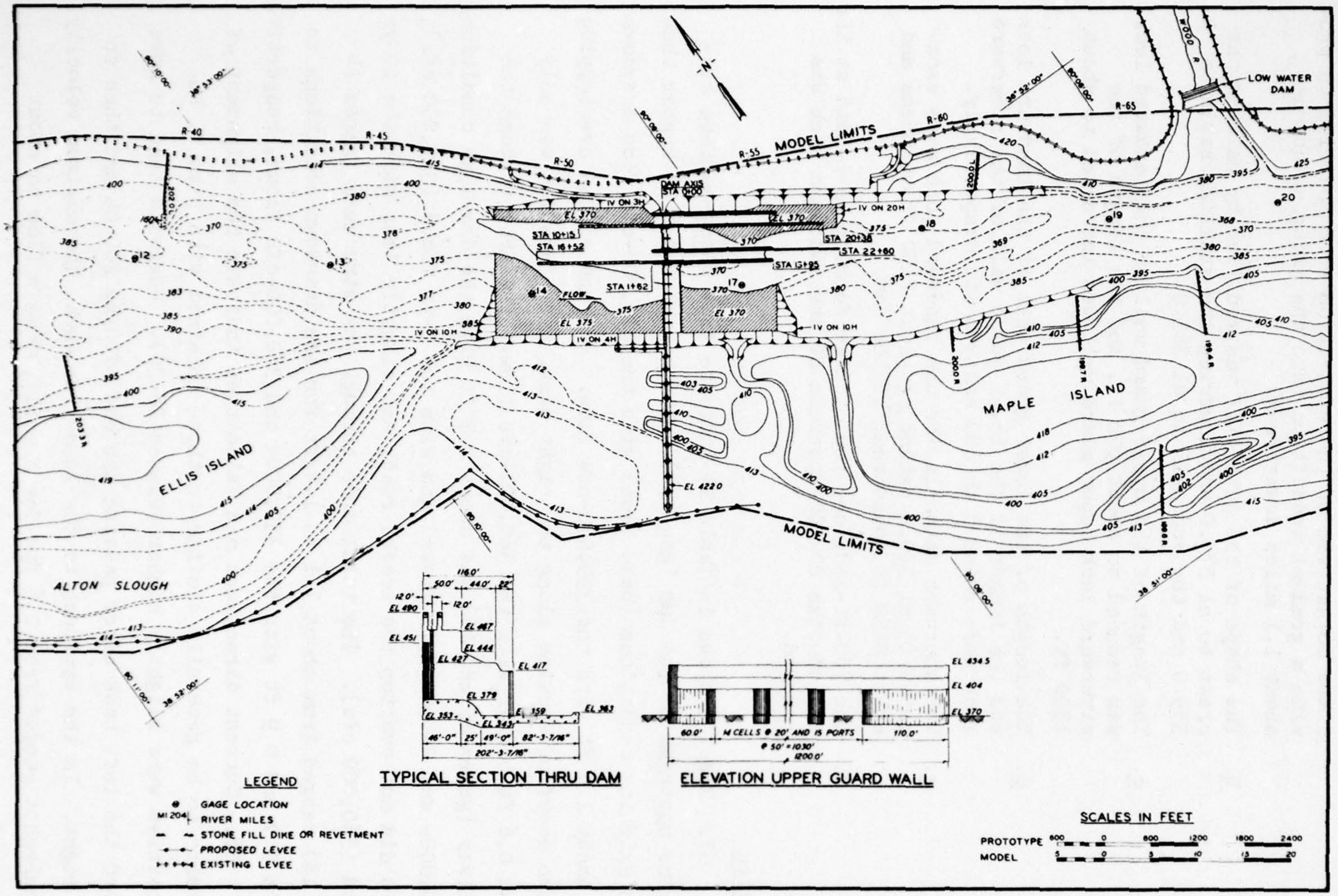


Figure 15. Plan E

dikes downstream of the excavation were modified to provide a gradual transition into the existing channel about 1.1 miles downstream.

b. The shape of the gate sill was modified by raising the crest to el 379.0 and lowering the stilling basin to el 359.0 and the end sill to el 363.0.

c. The length of the upper guard wall of the landward lock was reduced to about 900 ft, and the length of the riverward lock upper guard wall was increased to about 1500 ft.

d. The length of the lower guard wall of the landward lock and the length of the lower guide wall of the riverward lock were reduced to 600 and 900 ft, respectively.

e. The approach channels for the landward lock were excavated to el 370.0 extending about 2150 ft upstream and about 2250 ft downstream of the dam.

f. The 700-ft-long section of dike forming an L-head at the end of dike 202.0L upstream of the landward lock was removed.

Results

67. Results shown in Table 14 indicate that water-surface elevations upstream of the dam (gage 14) would be about 0.8 ft higher than with existing conditions (base test) with the 210,000-cfs flow drawdown and about 1.8 ft with the 339,000-cfs flow. Because of the overtopping of the overflow section along the right bank, the elevation was only about 0.6 ft higher with the 360,000-cfs flow. The drop through the spillway (gages 15 and 16) was about 0.3 ft with the drawdown conditions (210,000- and 240,000-cfs flows) and with the 5-yr flood (339,000 cfs), which did not overtop the access road, and slightly less with the 10-yr flood (360,000 cfs). The total drop through the structure (gages 14 and 18) ranged from about 1.1 to 1.4 ft for the drawdown conditions to about 1.9 and 0.9 ft with the 339,000- and 360,000-cfs flows, respectively.

68. Current directions and velocities indicate the alignment of currents to be generally similar to those obtained with plan D, but velocities were slightly higher (Plates 24-28). Currents tend to move toward the left bank after passing the end of dike 202.0L and then to the right. In the approach to the land-side lock, the maximum velocity of currents ranged from 1.6 fps for the 117,000-cfs flow to about

5.0 fps with the 5-yr flood. However, in the approach to the river-side lock, the maximum velocity of currents varied from about 2.3 fps with the 117,000-cfs flow to about 6.0 fps with the 5-yr flood. With the 10-yr flood (360,000 cfs) maximum velocities in the lock approaches were about 3.9 and 4.2 fps for the land-side and river-side locks, respectively. In the lower approach, currents were generally straighter than with plan D, moving from the spillway toward the left at considerably less angle, but velocities were higher. Velocities ranged from about 4.6 to 7.5 fps in the lower approach to the landward lock and from about 7.7 to about 10.4 fps in the approach to the riverward lock. The higher velocities in the lower lock approaches, in particular, were attributed to the decrease in the length of the spillway and in the amount of excavation of the channel bed. Considerable scouring of the bed can be expected along the right side of the channel downstream of the spillway, which should reduce velocities appreciably in the lower approach.

69. No serious navigation difficulties were indicated in the approaches to the locks, and two-way navigation should be feasible under most conditions. In the upper approach to the landward lock, tows would have to move close along the left bank since the upper guard wall tends to intercept less of the flow because of the reduction in the length of the wall. During low flows, the counterclockwise eddy would tend to slowly move the head of a downbound tow away from the guard wall. A fill along the left bank in the lock approach would provide a better guide for downbound tows and reduce the effects of the eddy. No difficulties were indicated in the lower approach to either lock with the proposed plan.

Plan E Cofferdam Tests

First-stage cofferdam

70. Description. The first-stage cofferdam was based on a reduction in the number of gate bays from eight to seven for plan E. It was essentially the same as for plan D except for changes in its size and shape, excavation along the right bank, and some modification in its

location with respect to the left bank (Plate 29). In the plan for the locks and dam structure, the gate bay on the Missouri side of the channel was eliminated and excavation of the bank was reduced by 125 ft along the axis of the dam. Excavation of the left bank was also modified by leaving most of the bank upstream of the axis of the dam as it exists with excavation downstream starting about 200 ft below the axis of the dam (same as for plan D cofferdam) with a transition in the bank line between the excavated and original bank (Plates 29 and 30). The new cofferdam extended 45 ft farther to the left than the original first-stage cofferdam and thus reduced the width of the navigation pass to about 550 ft depending on whether it is measured along the excavated or unexcavated bank. The access road from the cofferdam to the right bank levee was raised to el 422.0. On the upper end of the deflector, rock protection was placed 10 ft high with a 10-ft-wide crown extending along the deflector and the riverward face of the cofferdam.

71. Results. Water-surface elevations obtained with the cofferdam plan (Table 15) were about 0.3 to 0.4 ft higher than with plan D first-stage cofferdam plan (Table 11) with the 10- and 5-yr floods, respectively. Scouring of the bed as obtained in the model would lower water-surface elevations upstream of the cofferdam by about 0.5 to 0.6 ft with the flood flows (Table 16). The drop through the reach would be from 1.9 ft for the drawdown condition (240,000 cfs) to 1.1 ft for the 10-yr flood with no scouring of the bed between the cofferdam and left bank and from 0.3 to 0.6 ft with the bed scoured as developed in the model.

72. The scour pattern developed in the model with this cofferdam plan (Plate 29) indicates more extensive scouring than obtained with the original plan. Scouring was affected to some extent by the unexcavated portion of the left bank opposite the upper end of the main cofferdam. Maximum scour occurred about 200 ft riverward of the bend in the deflector with a bottom elevation of 317.0. Along the riverward face of the main cofferdam, scour reached a maximum depth at el 329.0 about 150 ft riverward of the cofferdam. The material scoured from the bed was deposited downstream and to the right of the riverward face of the

cofferdam with adequate channel depths for navigation maintained along the left bank.

73. Current directions and velocities obtained without scouring of the bed indicate currents through the pass to be reasonably straight with velocities affecting navigation about 1.0 fps higher than with the original first-stage cofferdam (Plate 29). The maximum velocities varying from about 7.3 to 7.7 fps would be somewhat lower with deepening of the channel through the pass. Spot velocities shown in Plates 30 and 31 indicate velocities up to 10.0 fps near the bend in the deflector and as much as 9.0 fps on the riverward side of the rock protection along the riverward arm of the main cofferdam. On the left bank velocities were relatively high ranging from about 5.9 to 7.8 fps along the bottom. Velocities through the pass between the cofferdam and the left bank were affected by the unexcavated portion of the left bank and the raising of the access road along the right bank. The distribution of flow and velocities within the navigation pass would be modified by scouring that can be expected between the cofferdam and left bank.

74. No serious navigation difficulties were indicated for upbound or downbound tows moving through the pass. There were also no tendencies for tows to be moved toward the cofferdam from either direction. Velocities would probably be somewhat higher than with the original cofferdam plan because of the reduction in the width of the pass. However, velocities affecting navigation would tend to be less than those indicated in Plate 29 with scouring of the channel bed.

Third-stage cofferdam.

75. Description. Tests were conducted to determine the effects that could be expected from modifying the third-stage cofferdam for the 7-gate dam (plan E), best sequence for construction, and conditions with various stages of construction of the cofferdam.

a. Phase 1 was the beginning of construction with the following conditions existing in the model:

(1) The riverward lock and spillway were installed based on plan E.

(2) The access road (overflow section) along the right bank between the spillway and levee with crest el 422.0 was maintained.

(3) The part of the second-stage cofferdam adjacent to the completed lock that would become part of the third-stage cofferdam was in place (Photo 2b).

b. Phase 2 assumed that construction of the cofferdam would begin on the river side and carry toward the Illinois side. This test was conducted with the upper arm of the cofferdam completed within 80 ft of the left bank.

c. Phase 3 assumed that construction of the upper arm of the cofferdam would be started on the Illinois bank and carried riverward. For this test, the upper arm of the cofferdam was completed within 85 ft of the existing portion of the second-stage cofferdam.

d. Phase 4 assumed that construction of the upper arm of the cofferdam would be started at the Illinois bank and at the remaining portion of the second-stage cofferdam at the same time. For this test, the upper arm of the cofferdam was completed except for an 85-ft section about halfway between the Illinois bank and the remaining portion of the second-stage cofferdam (Photo 2c).

e. Phase 5 assumed that the upper arm of the cofferdam was completed (Photo 2d).

76. Results. Water-surface elevations at gage 14 were from 0.2 to 0.9 ft higher than with existing conditions (base test) with the 360,000-cfs flow depending on the amount of cofferdam in place (Tables 17-21). The highest water-surface elevation at gage 14 with the upper arm of the cofferdam completed and all flow through the spillway and right overbank was 429.2 with the 10-yr flood. This is about 0.4 ft higher than with the third-stage cofferdam and the 8-gate spillway of plan D, which had the access road at el 414.0 instead of 422.0. The drop through the spillway (gages 15 and 16) was less than 0.3 ft, while the total drop through the structure (gages 14 and 18) with normal operation ranged from about 0.4 to 1.2 ft and from about 1.0 to 2.4 ft with the 240,000-cfs drawdown condition.

77. Current directions and velocities before start of construction of the third-stage cofferdam shown in Plate 32 indicate the alignment of currents to be generally straight and parallel to the alignment of the upper guard wall. With the lower flows of 75,000 and 240,000 cfs (drawdown), most of the flow in the approach to the completed lock moved

toward the left between the lock and left bank. With the 360,000-cfs flow, more of the flow within the lock approach would tend to move toward the upper guard wall. Maximum velocities in the upper lock approach ranged from about 2.9 to 4.4 fps with normal operation, and up to 5.5 fps with drawdown conditions and the 240,000-cfs flow. In the opening between the left bank and the remaining portion of the second-stage cofferdam, maximum velocities varied from about 5.3 to about 7.5 fps with normal operation and up to about 9.0 fps with drawdown and the 240,000-cfs flow. In the lower approach to the completed lock, the higher velocity currents were concentrated along the lock side of the channel during low flows, principally because of the shallow channel along the right side downstream of the spillway. Maximum velocities in the lower lock approach varied from about 4.3 to 6.3 fps with the flows tested. As construction of the upper arm of the third-stage cofferdam progresses, the amount of flow in the upper lock approach moving toward the upper guard wall of the completed lock would increase (Plates 33-36). In the approach to the lock, velocities were somewhat lower with the complete closure of the gap toward the left bank with an increase in flow moving toward the spillway side of the guard wall. Velocities of currents moving from the spillway toward the left bank across the lower approach to the lock were increased with the amount of closure of the upper arm of the cofferdam, reaching a maximum of 7.6 to about 10.8 fps with the complete closure.

78. Spot velocities obtained with the final gap in the upper arm of the cofferdam in various positions (Plates 37-39) indicate that the bottom velocities in the gap along the left bank would be lower than with the gap near the center or near the remaining end of the second-stage cofferdam. In all cases, maximum velocities were measured with the 240,000-cfs drawdown condition and were 8.9 fps along the left bank, 10.1 fps near the lock end, and 11.3 fps near the center.

79. No serious navigation difficulties were indicated for tows approaching or leaving the lock during construction of this stage cofferdam. Because of the currents moving from the approach toward the pass along the left bank with little or no flow toward the guard wall,

downbound tows would experience some difficulty, however, in approaching and remaining along the upper guard wall with the gap between the lock and left bank fully open, particularly with flows that do not substantially overtop the overflow section along the right bank.

80. As construction of the upper arm of the cofferdam progresses, flow through the ports would increase and navigation conditions for downbound tows would improve. With the closure between the lock and left bank substantially completed, tows could be made to drift into the upper lock approach from a considerable distance upstream. In general, navigation conditions during construction would improve with less of the cofferdam in place if the upper arm is constructed before the lower arm and if construction is started near the lock rather than along the left bank because of the effects on current in the lock approach. With the cofferdam closure nearing completion and before the pool is raised at the new structure, flow through the ports could present a hazard to small boats amd make it harder for tows to move away from the wall during low flows when the water-surface elevation is near or below the top of ports (el 404.0). The hazard could be reduced or eliminated by placing curtains extending below minimum water-surface elevation on at least half of the ports nearest the lock. High velocities could be encountered in the lower lock approach with the third-stage cofferdam, but no serious difficulties for navigation were indicated. Velocities in the approach and along the guide wall should be less than those indicated since considerable scouring and deepening of the channel to the right, away from the wall, can be expected.

PART IV: DISCUSSION OF RESULTS AND CONCLUSIONS

Limitation of Model Results

81. The analysis of the results of this investigation is based principally on a study of (a) various plans and modifications to determine the effects on water-surface elevations and current directions and velocities, and (b) the effects of resulting currents on the behavior of the model tow and towboat. In evaluating test results, consideration should be given to the fact that small changes in direction of flow or in velocities are not necessarily changes produced by a modification in plan since several floats introduced at the same point may follow a different path and move at slightly different velocities because of pulsating currents and eddies. The current directions and velocities shown in the plates were taken with floats submerged to a depth of a loaded barge (prototype) and are indicative of the currents that would affect the behavior of tows.

82. Because of the small model scale, it was difficult to reproduce accurately the hydraulic characteristics of the prototype structures or to measure water-surface elevations within an accuracy of ±0.1 ft prototype. The model was of the fixed-bed type and was not designed to simulate the movement of sediment in the prototype; therefore, changes in channel bed and banks that might be caused by the structures could not be developed naturally. A portion of the model was later converted to a movable bed to indicate scour patterns with various phases of construction. The model for these tests was adjusted by exaggerating the discharge and velocity scale until some movement of the bed material was obtained. Information was not available to permit any correlation of sediment movement in the model with that in the prototype or to simulate even approximately the composition of the material that would be encountered in the bed of the river. It was assumed in the model that the bed material in the prototype would consist of clean unconsolidated sand to the maximum depth of scour. Also, the scour pattern shown was developed with a series of extreme flows only, including the 5- and

10-yr floods and maximum drawdown condition. The results of scour tests, therefore, are highly qualitative and were developed to provide some general indications of the scour and fill patterns that could be expected with the conditions imposed. Changes in the elevations of the bed should not be considered as any indication of the depth of scour that can be expected in the river and are provided only to illustrate the relative effects of various plans based on the flows reproduced. Furthermore, current direction and velocity measurements were made with no modifications in the bed; scouring of the bed, which can be expected with some of the plans, would produce changes in the distribution of flow and current velocities. Modifications in the channel bed brought about by installation of various plans would cause water-surface elevations to tend to be lower than indicated by test results.

Summary of Results and Conclusions

83. The following results and conclusions were developed for the existing site from the investigation:

- a. During high flows, navigation conditions at the existing locks could be extremely difficult and hazardous because of the alignment of the currents moving across the approach to the locks toward the spillway.
- b. Satisfactory navigation conditions for two-way traffic could be produced by construction of new locks with a gate bay between the locks and between the landward lock and the left descending bank line similar to plan A. Tests at the existing site were limited because of the deteriorated condition of the existing spillway, complexity of the arrangement required, and difficulty of maintaining traffic during construction.

84. The following results and conclusions were developed for the proposed site from these investigations:

- a. Two-way navigation could be provided by a replacement structure located about 2 miles downstream of the existing structure. A satisfactory plan included two parallel 1200-ft locks separated 265 ft with two 110-ft gates between locks located along the left bank as in plan D. Reducing the number of 110-ft gates in the spillway from eight to seven, as in plan E, would not seriously affect navigation conditions in the lock approaches.

b. The replacement structure about 2 miles downstream would require: (1) the modification of the existing bridges; (2) the removal of at least 11 of the existing gate bays and piers adjacent to the existing locks; and (3) the removal of the existing locks and lock walls except for the intermediate lock wall and the landward wall of the existing 600-ft lock. Navigation conditions through the bridges at the existing site would be better with the entire existing intermediate lock wall in place and with a downstream extension of the existing 600-ft lock guide wall (plan B). With the modifications mentioned, two-way traffic through the reach is practical, and small tows and pleasure crafts could use the opening between the intermediate wall and the landward wall of the existing 600-ft lock.

c. Installation of one lock on each side of the channel at the replacement site was considered during the study but was discarded because of the difficulty of maintaining a channel along the right side and of developing currents suitable for navigation. Also, tows approaching or leaving the upper approach to the lock along the right bank would have to cross the river and become properly aligned within a relatively short distance to approach the lock or move through the bridge spans.

d. Navigation conditions in the upper approach for downbound tows using the land-side lock with plan E were not as good as those with plan D because of the shift of the locks and the dam downstream and the shorter upper guard wall. However, conditions could be improved by filling along the left bank opposite the guard wall to an elevation of at least 420.0 and by raising the top of the ports in the guard wall to increase flow landward of the guard wall to reduce the size of the eddy and the tendency for tows to be moved away from the wall by the eddy. Ports would be required in the upper guard walls to reduce crosscurrents near the ends of the walls and provide currents that would assist tows in moving toward the lock side of the walls.

e. Navigation conditions in the upper approaches to the locks would be better with a 1500-ft-long upper guard wall on the river-side lock and a 1200-ft-long guard wall on the land-side lock. However, no serious navigation difficulties were indicated with the shorter guard walls of plan E.

f. Gates will be required between the locks with lock separation to eliminate serious crosscurrents in the lock approaches. Satisfactory navigation conditions could be maintained with one 110-ft gate between the locks, but to assure that at least one gate will always be in operation two gates are recommended. Operating the gates in between

the locks the same as those in the spillway would provide satisfactory navigation conditions.

g. A dike or fill extending from the right abutment or overflow section would be required along the right top bank to prevent flood flows that do not substantially overtop the overflow section from moving toward the spillway parallel to the axis of the dam creating a disturbance in front of the gate bays near the right abutment and reducing flow through those gates. The dike should be at least 600 ft long with the top elevation the same as that of the overflow section (422.0).

h. Currents moving from the spillway toward the left bank will tend to affect tows in the lower lock approach. The effects of these currents are reduced by flow through the gates between the locks. Scouring can be expected along the right side of the channel downstream of the spillway which would reduce velocities to less than those shown in the results of tests and improve the alignment of currents in the lower approach to the locks.

i. Dredging and improvement of the alignment of the left bank downstream of the landward lock would be required with plan E to provide a straight and unobstructed approach for upbound tows using that lock, particularly with the short guide wall.

j. Some increase in water-surface elevations can be expected upstream of the existing structure with any of the plans tested. With the seven-gate spillway of plan E, which had a higher gate sill, water-surface elevations were considerably higher than with the eight-gate spillway of plan D. Closure of one of the gates between the locks could raise stages upstream of the dam from 0.1 to 0.2 ft with plan D and even more so with plan E. Stages with plan E were also affected by the limited dredging downstream of the spillway and should be lower than indicated by the model results as the channel downstream becomes deeper with the plan.

k. During construction of the project, a two-stage cofferdam with a bypass canal along the right bank was not considered practical because of navigation difficulties for tows using the canal and the problems involved in maintaining the canal.

l. Satisfactory navigation conditions could be maintained through the reach using a three-stage cofferdam plan starting near the right bank. Considerable scouring can be expected near the upstream riverward corner of the cofferdam. Use of a deflector, as developed during the tests, would move the maximum scour away from the main cofferdam and tend to reduce the depth of scour.

m. Navigation conditions for downbound tows using the completed lock would be better during construction of the third-stage cofferdam if the upper arm of the cofferdam is constructed before or at the same time as the lower arm. Also, beginning construction of the cofferdam near the lock would provide better navigation conditions than if started at the left bank.

n. Curtains would be required on the ports in the lower half of the upper guard wall to protect small boats using the lock during construction of the third-stage cofferdam before the pool is raised at the new structure. The bottom of the curtains should be at least 2 to 3 ft below the minimum pool.

o. Considerable scouring of the bed below the spillway could be expected with only one gate full or half open and a head of 24 ft. The scouring could endanger the structure unless adequately protected.

Table 1

Base Test Water-Surface Elevations

Gage No.	Water-Surface Elevation, ft msl, at Indicated Discharge, cfs					
	117,000	210,000	312,000	339,000	545,000	650,000
1	419.1	414.8	421.8	419.5	433.3	444.3
2	419.1	414.7	421.7	419.4	433.2	444.2
3	419.1	414.4	421.4	419.0	433.0	444.2
4	419.0	414.2	421.3	418.7	432.9	444.1
5	419.0	414.0	421.0	418.5	432.8	444.0
6	418.9	413.8	420.7	418.2	432.5	443.9
6-A	418.9*	414.0*	420.8	418.1*	432.6*	444.0
7-8	418.8	413.7	420.7	418.1	432.4	443.8
9-10	403.3	413.6	420.6	418.0	432.1	443.6
11	403.2*	413.6	420.5*	417.9	432.0*	443.4*
12	402.9	413.4	420.3	417.6	431.9	443.4
13	402.8	413.2	420.1	417.4	431.8	443.3
14	402.5	413.0	420.0	417.2	431.6	443.2
15	402.4	412.9	419.9	417.1	431.5	443.1
16	402.4	412.9	419.9	417.1	431.5	443.1
17	402.2	412.7	419.8	416.9	431.4	443.0
18	401.9	412.5	419.6	416.7	431.2	442.9
19	401.6	412.3	419.2	416.2	430.8	442.6
20	401.6	412.2	419.0	416.0	430.6	442.4

* Controlled elevation.

Table 2

Plan A Water-Surface Elevations

Gage No.	Water-Surface Elevation, ft msl, at Indicated Discharge, cfs 117,000	210,000	339,000	545,000
1	419.1	415.0	420.0	433.7
2	419.1	414.9	419.9	433.6
3	419.1	414.7	419.5	433.4
4	419.0	414.5	419.3	433.3
5	419.0	414.4	419.2	433.2
6	418.9*	414.0	418.6	432.8
7-8	418.8	413.7	418.5	432.6
9-10	403.5	413.5	418.2	432.1
11	403.5	413.5	418.1	431.9
12	403.1	413.3	417.6	431.8
13	403.0	413.1	417.4	431.7
14	402.4	412.9	417.2	431.5
15	402.4	412.8	417.1	431.4
16	402.4	412.8	417.1	431.4
17	402.2	412.7	416.9	431.3
18	401.9	412.5	416.7	431.1
19	401.6	412.3	416.2	430.8
20	401.6*	412.2*	416.0*	430.6*

Note: Gage 6-A was affected by the rock dike upstream and thus not included. Gates between the locks open 15 ft with open-river conditions.

* Controlled elevation.

Table 3

Plan B Water-Surface Elevations

Gage No.	Water-Surface Elevation, ft msl, at Indicated Discharge, cfs 117,000	210,000	339,000	545,000
1	419.1	415.0	420.0	433.7
2	419.1	414.9	419.9	433.6
3	419.1	414.6	419.5	433.4
4	419.1	414.4	419.3	433.3
5	419.0	414.2	419.1	433.1
6	419.0	414.1	418.9	432.9
6-A	418.8	414.2	419.0	432.9
7-8	419.0	414.1	418.8	432.8
9-10	419.0	413.9	418.7	432.6
11	418.9	413.9	418.6	432.5
12	418.9	413.7	418.4	432.4
13	418.9	413.5	418.2	432.3
14	418.8	413.2	417.6	431.9
14-A	418.9*	413.5	418.1	432.4
15	418.8	413.1	417.4	431.8
16	402.3	413.0	417.3	431.7
17	402.2	412.9	417.1	431.4
18	401.9	412.6	416.6	431.1
19	401.6	412.4	416.2	430.8
20	401.6*	412.2*	416.0*	430.6*

Note: Gate openings were the same for all gates including those between the locks.

* Controlled elevation.

Table 4

Plan B-1 Water-Surface Elevations

Gage No.	Water-Surface Elevation, ft msl, at Indicated Discharge, cfs 117,000	210,000	339,000	545,000
1	419.0	414.9	419.7	433.4
2	419.0	414.8	419.6	433.3
3	419.0	414.5	419.3	433.2
4	419.0	414.3	419.1	433.1
5	418.9	414.1	418.8	432.9
6	418.9	413.9	418.6	432.8
6-A	418.8	413.9	418.6	432.7
7-8	418.9	413.9	418.6	432.7
9-10	418.8	413.8	418.5	432.5
11	418.8	413.7	418.4	432.4
12	418.8	413.6	418.2	432.3
13	418.8	413.4	418.0	432.2
14	418.7	413.1	417.5	431.8
14-A	418.9*	413.6	417.9	432.2
15	418.7	413.1	417.4	431.7
16	402.2	413.0	417.3	431.6
17	402.1	412.9	417.1	431.4
18	401.8	412.6	416.6	431.0
19	401.6	412.3	416.2	430.8
20	401.6*	412.2*	416.0*	430.6*

Note: Gate openings were the same for all gates including those between locks.

* Controlled elevation.

Table 5

Plan C Water-Surface Elevations

Gage No.	Water-Surface Elevation, ft msl, at Indicated Discharge, cfs 117,000	210,000	339,000	545,000
1	419.2	414.9	419.8	433.6
2	419.2	414.8	419.7	433.5
3	419.2	414.5	419.3	433.3
4	419.1	414.3	419.1	433.1
5	419.1	414.1	418.9	433.0
6	419.1	413.9	418.7	432.7
6-A	419.1	413.9	418.7	432.7
7-8	419.1	413.9	418.6	432.7
9-10	419.1	413.8	418.5	432.5
11	419.0	413.8	418.5	432.4
12	419.0	413.5	418.1	432.3
13	418.9	413.4	418.0	432.2
14	418.9	413.1	417.5	431.8
14-A	418.9*	413.3	417.7	432.0
15	418.9	413.1	417.5	431.8
16	402.3	413.1	417.4	431.8
17	402.2	412.9	417.1	431.5
18	401.8	412.6	416.7	431.0
19	401.6	412.3	416.1	430.8
20	401.6*	412.2*	416.0*	430.6*

* Controlled elevation.

Table 6

Plan D Water-Surface Elevations (with One and Two Gates Between Locks)

	Water-Surface Elevation, ft msl, at Indicated Discharge, cfs								
	117,000	210,000		339,000		545,000		650,000	
Gage No.	Controlled Riverflow	Two Gates	One Gate*	Two Gates	One Gate*	Two Gates	One Gate*	Two Gates	One Gate*
1	419.3	414.8	414.9	419.6	419.6	433.5	433.7	444.4	444.5
2	419.2	414.7	414.8	419.5	419.5	433.4	433.6	444.3	444.5
3	419.2	414.4	414.5	419.1	419.2	433.2	433.4	444.3	444.3
4	419.1	414.2	414.3	418.8	419.0	433.1	433.3	444.2	444.3
5	419.1	414.0	414.1	418.6	418.8	432.9	433.1	444.1	444.2
6	419.0	413.8	413.9	418.4	418.6	432.7	432.9	444.0	444.1
7-8	419.0	413.8	413.9	418.3	418.5	432.6	432.8	443.9	444.0
9-10	419.0	413.7	413.8	418.2	418.4	432.4	432.6	443.8	443.9
11	419.0	413.7	413.8	418.1	418.3	432.3	432.5	443.7	443.8
12	419.0	413.5	413.6	417.8	418.0	432.2	432.4	443.6	443.7
13	418.9	413.3	413.4	417.7	417.8	432.1	432.3	443.5	443.6
14	418.9**	413.2	413.3	417.4	417.6	431.9	432.1	443.4	443.5
15	418.9	413.0	413.0	417.0	417.0	431.6	431.7	443.4	443.3
16	405.0	413.0	413.0	417.0	417.0	431.6	431.7	443.2	443.3
17	402.3	412.8	412.8	416.9	416.9	431.4	431.4	443.0	443.0
18	402.1	412.6	412.6	416.7	416.7	431.0	431.0	442.8	442.8
19	401.6	412.3	412.3	416.2	416.2	430.7	430.7	442.5	442.5
20	401.6**	412.2**	412.2**	416.0**	416.0**	430.6**	430.6**	442.4**	442.4**

* Gate between the locks adjacent to the riverward lock closed.
** Controlled elevation.

Table 7

Plan D Distribution of Flow Through Dam

(with One and Two Gates Between Locks)

Gate No.*	Percent of Total Flow at Indicated Discharge, cfs: 210,000 Two Gates	210,000 One Gate	545,000 Two Gates	545,000 One Gate
1L	9.1	12.8	7.9	11.3
2L	9.5	Closed	8.6	Closed
1	6.0	5.4		
2	9.5	10.2		
3	10.2	11.0		
4	11.6	12.2		
5	11.4	12.6		
6	11.6	12.6		
7	11.6	12.6		
8	9.5	10.6		

* Gates numbered from left to right; 1L and 2L located between the locks.

Table 8

Plan D Velocities at Discharge of 134,000 cfs

(with One Spillway Gate Fully Open)

Station*	Velocity,** fps, at Indicated Distance (ft) Riverward of Lock Wall					
	Gate 1			Gate 3		
	10	50	100	250	300	350
1+85B (end sill)	23.9	25.0	24.1	24.3	25.5	25.3
2+80B	20.6	17.2	10.8	12.1	21.0	18.7
4+80B	19.1	13.6	11.5	14.7	18.6	13.5
6+80B	17.3	18.6	20.2	18.6	21.8	14.1
8+30B	13.7	15.8	18.8	14.8	16.2	13.5
11+50B	13.4	13.7	12.8	11.3	10.4	11.3
15+00B	8.2	10.4	11.2	8.1	9.6	8.3

Note: Upper pool el 419.0 and lower pool el 395.0.
* Axis of dam at sta 0+00.
** Velocities measured near channel bed.

Table 9

Plan D Velocities at Discharge of 91,000 cfs

(with One Spillway Gate Open 12 ft)

Station*	Velocity,** fps, at Indicated Distance (ft) Riverward of Lock Wall					
	Gate 1			Gate 3		
	10	50	100	250	300	350
1+85B (end sill)	24.5	27.6	21.2	28.4	26.0	16.6
2+80B	19.8	19.3	15.0	17.4	21.0	18.1
4+80B	20.9	19.8	16.8	17.1	18.1	14.7
6+80B	17.1	16.3	16.0	13.9	15.5	8.3
8+30B	16.4	15.9	16.0	14.6	13.6	10.4
11+50B	13.5	14.3	13.0	12.6	12.6	12.2
15+00B	10.6	12.7	12.6	10.9	11.9	11.8

Note: Upper pool el 419.0 and lower pool el 395.0.
* Axis of dam at sta 0+00.
** Velocities measured near channel bed.

Table 10

Water-Surface Elevations for Plan D Cofferdam

Test Base Conditions

Gage No.	Water-Surface Elevation, ft msl, at Indicated Discharge, cfs 210,000	315,000	360,000
1	415.3	426.5	429.3
2	415.2	426.4	429.3
3	414.9	426.3	429.1
4	414.7	426.2	429.0
5	414.5	426.1	428.9
6	414.4	426.0	428.9
7-8	414.4	426.0	428.8
9-10	414.3	425.9	428.7
11	414.2	425.9	428.6
12	414.0	425.7	428.5
13	413.8	425.6	428.4
14	413.7	425.5	428.3
15	413.7	425.5	428.3
16	413.7	425.5	428.2
17	413.6*	425.5*	428.2*
18	413.4	425.4	428.1
19	413.1	425.3	427.9
20	413.0	425.2	427.8

* Controlled elevations.

Table 11

Water-Surface Elevations for Plan D First-Stage

Cofferdam Test

Gage No.	Water-Surface Elevation, ft msl, at Indicated Discharge, cfs 210,000	240,000	315,000	360,000
1	416.1	416.7	427.2	429.9
2	416.0	416.6	427.1	429.9
3	415.7	416.3	427.0	429.8
4	415.6	416.1	427.0	429.7
5	415.4	416.0	426.9	429.6
6	415.2	415.8	426.7	429.5
7-8	415.2	415.8	426.7	429.4
9-10	415.1	415.6	426.5	429.3
11	415.0	415.4	426.4	429.2
12	414.9	415.3	426.3	429.1
13	414.6	415.0	426.3	429.0
14	414.5	414.8	426.2	429.0
15	--	--	--	--
16	--	--	--	--
17	413.4	413.3	425.5	428.2
18	413.3	413.2	425.5	428.2
19	413.2	413.1	425.4	428.1
20	413.0*	412.8*	425.2*	427.8*

Note: Test made with access road raised to el 414.0. Data taken with the model bed restored to its original condition without any scouring that would develop with the cofferdam in place.

* Controlled elevation.

Table 12

Water-Surface Elevations for Plan D Second-Stage Cofferdam Test

Gage No.	Water-Surface Elevation, ft msl, at Indicated Discharge, cfs 240,000	315,000	360,000
1	416.3	426.9	429.5
2	416.2	426.9	429.5
3	415.9	426.6	429.4
4	415.7	426.6	429.3
5	415.5	426.5	429.2
6	415.2	426.4	429.1
7-8	415.2	426.4	429.0
9-10	415.1	426.3	428.9
11	415.0	426.2	428.9
12	414.7	426.1	428.8
13	414.5	426.0	428.7
14	414.3	426.0	428.6
15	414.2	425.8	428.5
16	413.7	425.6	428.3
17	--	--	--
18	413.5	425.5	428.2
19	413.2	425.3	428.0
20	412.8*	425.2*	427.8*

Note: Test made with access road raised to el 414.0. Data taken with model bed restored to its original condition without any scouring that would develop with the cofferdam in place.

* Controlled elevation.

Table 13

Water-Surface Elevations for Plan D Third-Stage

Cofferdam Test

Gage No.	Water-Surface Elevation, ft msl, at Indicated Discharge, cfs 240,000	315,000	360,000
1	416.5	427.3	429.8
2	416.3	427.2	429.8
3	416.0	427.1	429.7
4	415.7	427.0	429.7
5	415.6	426.9	429.5
6	415.4	426.8	429.3
7-8	415.3	426.7	429.3
9-10	415.2	426.6	429.2
11	415.1	426.5	429.1
12	414.8	426.4	429.1
13	414.6	426.4	428.9
14	414.4	426.3	428.8
15	414.1	426.1	428.6
16	413.7	425.8	428.4
17	413.5	425.6	428.2
18	413.4	425.5	428.2
19	413.1	425.4	428.0
20	412.8*	425.2*	427.8*

Note: Test made with access road raised to el 414.0. Data taken with model bed restored to its original condition without any scouring that would develop with the cofferdam in place.

* Controlled elevation.

Table 14

Plan E Water-Surface Elevations

Gage No.	Water-Surface Elevation, ft msl, at Indicated Discharge, cfs 117,000	210,000	240,000	339,000	360,000
1	419.1	416.0	416.7	420.6	430.0
2	419.1	415.8	416.5	420.5	429.9
3	419.1	415.5	416.2	420.1	429.7
4	419.1	415.4	416.0	420.0	429.6
5	419.0	415.2	415.8	419.8	429.6
6	419.0	415.1	415.6	419.6	429.4
7-8	419.0	415.1	415.6	419.6	429.4
9-10	418.9	415.0	415.4	419.4	429.2
11	418.9	414.9	415.3	419.3	429.1
12	418.9	414.7	415.2	419.0	429.1
13	418.9	414.7	414.8	419.0	429.0
14	418.8	414.5	414.7	418.6	428.9
15	418.8*	414.2	414.4	418.0	428.6
16	403.4	413.9	414.1	417.7	428.4
17	403.5	414.1	414.1	417.7	428.4
18	401.9	413.4	413.3	416.7	428.0
19	401.7	413.2	413.0	416.2	428.0
20	401.6*	413.0*	412.8*	416.0*	427.8*

* Controlled elevation.

Table 15

Water-Surface Elevations for Plan E First-Stage Cofferdam Test

Gage No.	Water-Surface Elevation, ft msl, at Indicated Discharge, cfs 210,000	240,000	315,000	360,000
1	416.3	417.0	427.5	430.1
2	416.2	416.9	427.5	430.1
3	416.0	416.6	427.3	430.0
4	415.8	416.5	427.2	429.9
5	415.7	416.3	427.2	429.8
6	415.6	416.1	427.1	429.7
7-8	415.5	416.1	427.0	429.6
9-10	415.4	416.0	426.9	429.5
11	415.3	415.8	426.9	429.5
12	415.2	415.7	426.8	429.5
13	415.0	415.4	426.7	429.4
14	414.9	415.3	426.6	429.3
15	*	*	*	*
16	*	*	*	*
17	413.4	413.4	425.4	428.2
18	413.3	413.3	425.4	428.1
19	413.3	413.2	425.3	428.1
20	413.0**	412.8**	425.2**	427.8**

Note: Test made with access road raised to el 422.0. Data taken with the model bed restored to its original condition without any scouring that would develop with the cofferdam in place.

* Gages inside cofferdam.

** Controlled elevation.

Table 16

Water-Surface Elevations for Plan E First-Stage Cofferdam Test

Gage No.	Water-Surface Elevation, ft msl, at Indicated Discharge, cfs 210,000	240,000	315,000	360,000
1	415.5	416.1	427.0	429.7
2	415.4	416.0	427.0	429.6
3	415.2	415.8	426.8	429.5
4	415.0	415.4	426.7	429.4
5	414.9	415.3	426.6	429.3
6	414.8	415.1	426.5	429.2
7-8	414.7	415.1	426.5	429.1
9-10	414.6	415.0	426.4	429.0
11	414.6	414.9	426.4	429.0
12	414.4	414.7	426.3	428.9
13	414.2	414.4	426.2	428.8
14	414.1	414.3	426.1	428.7
15	--	--	--	--
16	--	--	--	--
17	413.8	413.7	425.5	428.1
18	413.6	413.6	425.4	428.1
19	413.4	413.2	425.4	428.0
20	413.0*	412.8*	425.2*	427.8*

Note: Test made with access road raised to el 422.0. Data taken with model bed scoured based on the effects of the cofferdam in the model.

* Controlled elevation.

Table 17

Water-Surface Elevations for Plan E Third-Stage Cofferdam Test (Phase 1*)

Gage No.	Water-Surface Elevation, ft msl, at Indicated Discharge, cfs 75,000	240,000	360,000
1	419.1	416.2	429.6
2	419.1	416.1	429.5
3	419.0	415.8	429.4
4	419.0	415.7	429.3
5	419.0	415.5	429.2
6	419.0**	415.3	429.1
7-8	418.8	415.2	429.0
9-10	402.9	415.1	428.9
11	402.9	415.0	428.9
12	402.8	414.8	428.8
13	402.6	414.5	428.7
14	402.6	414.4	428.5
15	402.6	414.2	428.4
16	402.6	414.1	428.3
17	402.6	414.0	428.2
18	402.1	413.4	428.1
19	402.0	413.0	427.9
20	402.0**	412.8**	427.8**

Note: Test made with access road raised to el 422.0. Data taken with model bed restored to its original condition without any scouring that would develop with the cofferdam in place.

* Start of construction of third stage of cofferdam.

** Controlled elevation.

Table 18

Water-Surface Elevations for Plan E Third-Stage Cofferdam Test (Phase 2*)

Gage No.	Water-Surface Elevation, ft msl, at Indicated Discharge, cfs 75,000	240,000	360,000
1	419.1	416.9	430.1
2	419.1	416.9	430.0
3	419.0	416.6	429.9
4	419.0	416.5	429.8
5	419.0	416.3	429.7
6	419.0**	416.2	429.6
7-8	418.8	416.1	429.6
9-10	403.5	415.9	429.4
11	403.5	415.9	429.3
12	403.4	415.8	429.3
13	403.3	415.5	429.2
14	403.2	415.3	429.1
15	403.2	414.7	428.7
16	403.2	414.4	428.4
17	403.0	414.4	428.4
18	402.2	413.3	428.1
19	402.1	413.0	428.0
20	402.0**	412.8**	427.8**

Note: Test made with access road raised to el 422.0.
* Upper arm of cofferdam completed except for 80-ft section near left bank.
** Controlled elevation.

Table 19

Water-Surface Elevations for Plan E Third-Stage

Cofferdam Test (Phase 3*)

Gage No.	Water-Surface Elevation, ft msl, at Indicated Discharge, cfs 75,000	240,000	360,000
1	419.1	416.9	430.0
2	419.1	416.8	429.9
3	419.0	416.5	429.8
4	419.0	416.4	429.7
5	419.0	416.2	429.7
6	419.0**	416.0	429.5
7-8	419.0	416.0	429.5
9-10	403.3	415.8	429.3
11	403.2	415.8	429.3
12	403.2	415.6	429.3
13	403.1	415.3	429.2
14	403.0	415.2	429.0
15	403.0	414.6	428.6
16	402.9	414.4	428.4
17	402.9	414.3	428.3
18	402.2	413.4	428.0
19	402.1	413.0	427.9
20	402.0**	412.8**	427.8**

Note: Test made with access road raised to el 422.0.

* Upper arm of cofferdam completed except for 85-ft section near locks.

** Controlled elevation.

Table 20

Water-Surface Elevations for Plan E Third-Stage

Cofferdam Test (Phase 4*)

Gage No.	Water-Surface Elevation, ft msl, at Indicated Discharge, cfs 75,000	240,000	360,000
1	419.1.	416.9	430.0
2	419.1	416.8	429.9
3	419.0	416.4	429.8
4	419.0	416.3	429.7
5	419.0	416.1	429.7
6	419.0**	416.0	429.5
7-8	419.0	415.9	429.5
9-10	403.3	415.8	429.3
11	403.3	415.8	429.3
12	403.2	415.5	429.3
13	403.1	415.2	429.2
14	403.0	415.2	429.0
15	403.0	414.6	428.6
16	403.0	414.3	428.4
17	402.9	414.2	428.3
18	402.2	413.4	428.0
19	402.1	413.0	427.9
20	402.0**	412.8**	427.8**

Note: Test made with access road raised to el 422.0.

* Upper arm of cofferdam completed except for 85-ft section near center.

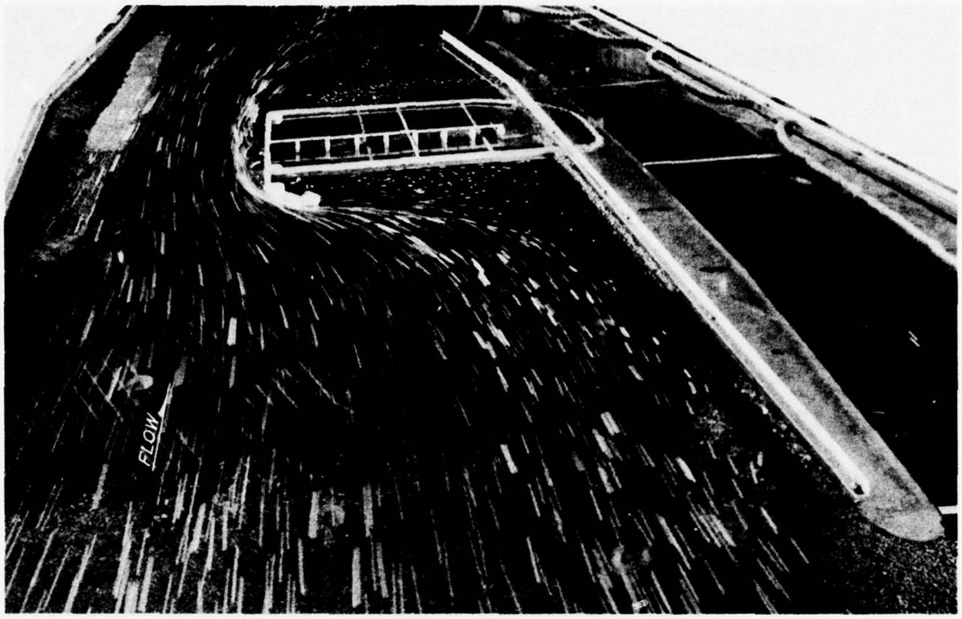
** Controlled elevation.

Table 21

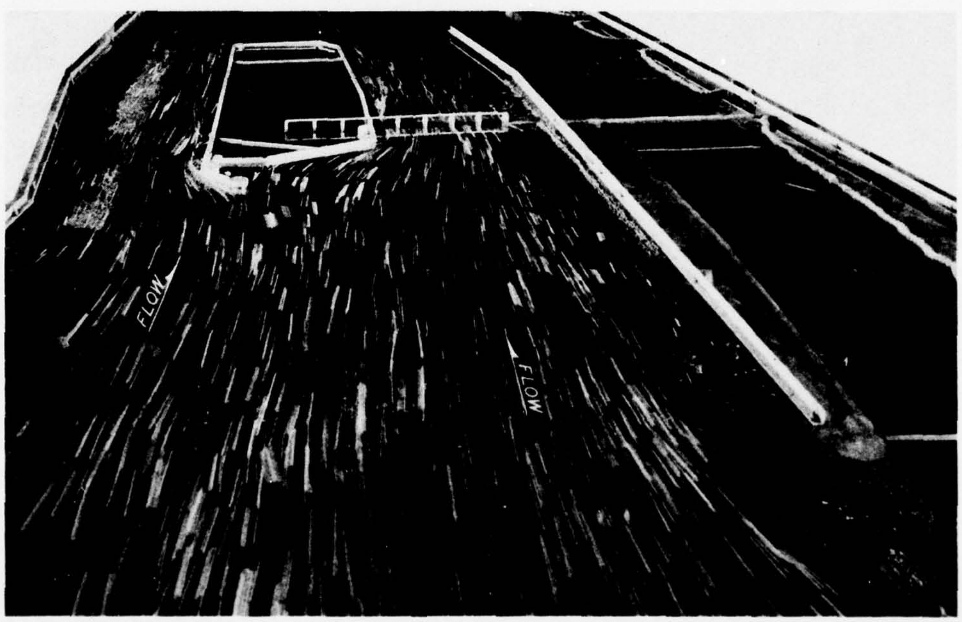
Water-Surface Elevations for Plan E Third-Stage Cofferdam Test (Phase 5*)

Gage No.	Water-Surface Elevation, ft msl, at Indicated Discharge, cfs		
	75,000	240,000	360,000
1	419.1	417.1	430.2
2	419.1	417.1	430.1
3	419.0	416.9	430.0
4	419.0	416.8	430.0
5	419.0	416.6	429.8
6	419.0**	416.4	429.8
7-8	418.9	416.4	429.7
9-10	403.5	416.2	429.5
11	403.5	416.2	429.5
12	403.4	416.0	429.5
13	403.3	415.7	429.3
14	403.3	415.6	429.2
15	403.2	414.8	428.6
16	403.1	414.5	428.3
17	403.0	414.5	428.3
18	402.1	413.2	428.0
19	402.0	413.0	427.9
20	402.0**	412.8**	427.8**

Note: Test made with access road raised to el 422.0
* Construction of third stage of cofferdam completed.
** Controlled elevation.

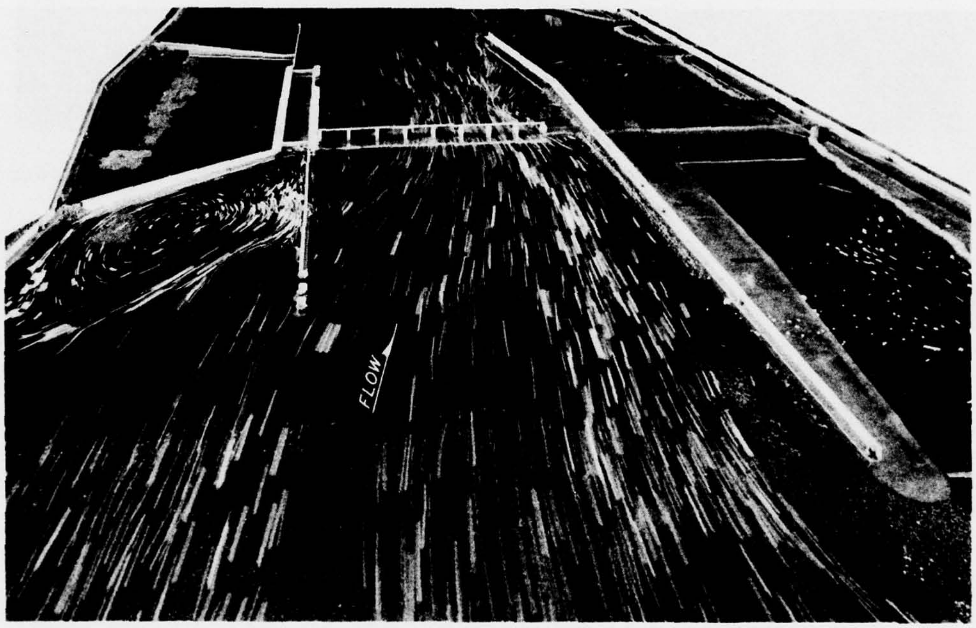


a. First stage: surface currents as affected by deflector at upper corner of cofferdam



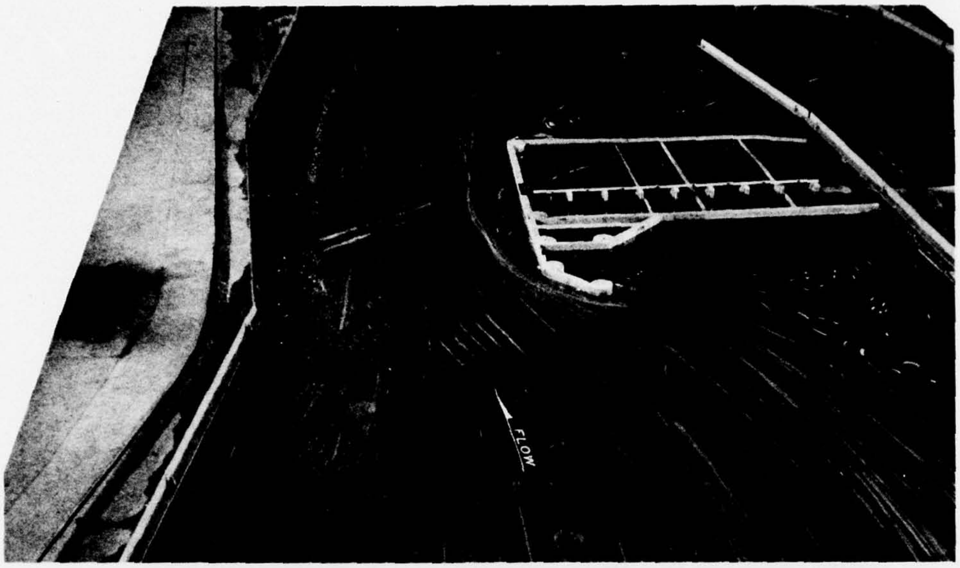
b. Second stage: surface currents through navigation pass between cofferdam and left bank with deflector on upper corner of cofferdam and through spillway

Photo 1. Plan D cofferdam tests; discharge 210,000 cfs drawdown (sheet 1 of 2)

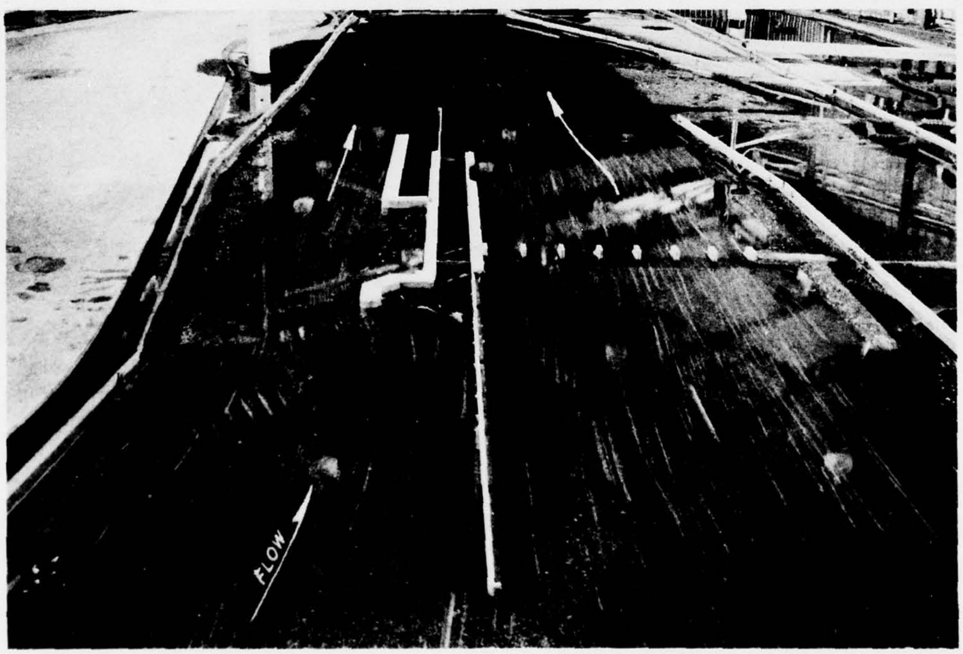


c. Third stage: surface currents in lock approach and through completed spillway; eddy upstream of cofferdam exaggerated because of the effects of surface tension

Photo 1 (sheet 2 of 2)

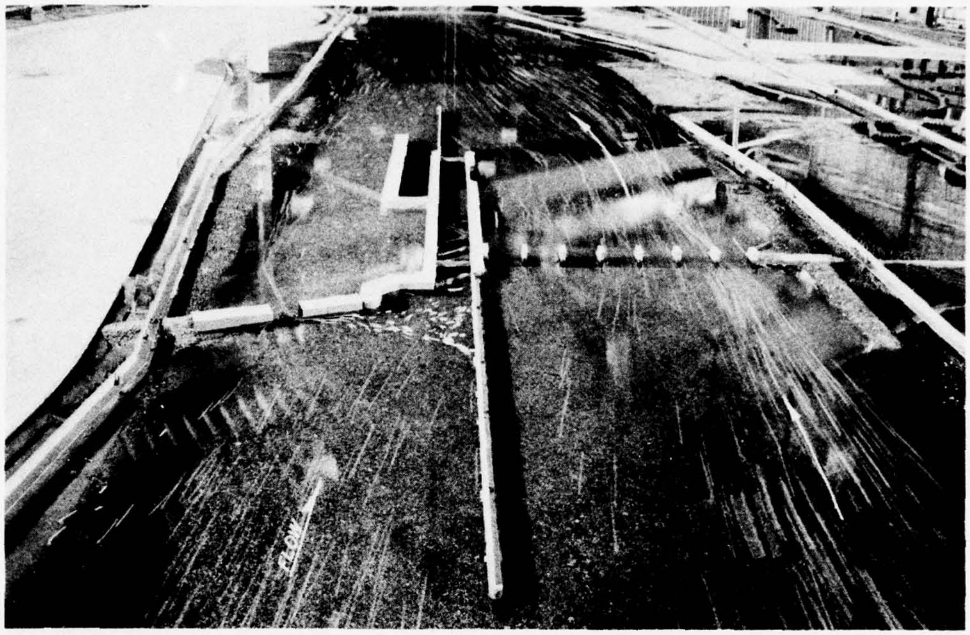


a. First stage: surface currents through navigation pass between cofferdam and left bank. (Note effect of deflector on currents near the cofferdam)

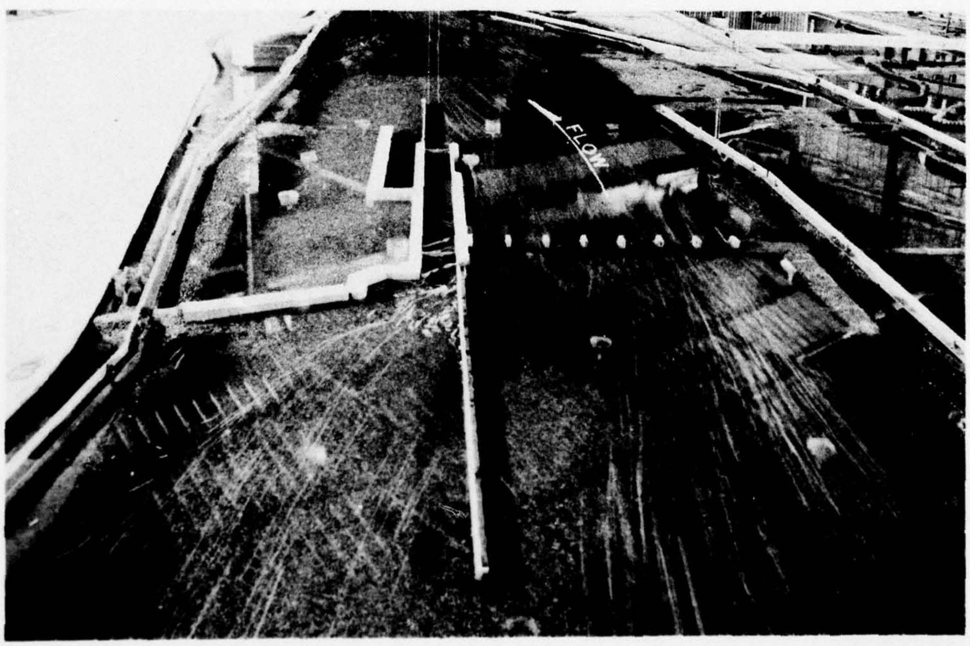


b. Third stage (phase 1): surface currents through the completed spillway, in the approaches to the completed lock and through the pass between the remainder of the second-stage cofferdam and the left bank. (Note start of construction)

Photo 2. Plan E cofferdam tests; discharge of 210,000 cfs drawdown (sheet 1 of 2)



c. Third stage (phase 4): surface currents through spillway and in the approaches to the completed lock. (Note upper arm of cofferdam completed except for 85-ft section near center of pass between lock and left bank)



d. Third stage (phase 5): surface currents through spillway and in the approaches to the lock. (Note upper arm of cofferdam completed)

Photo 2 (sheet 2 of 2)

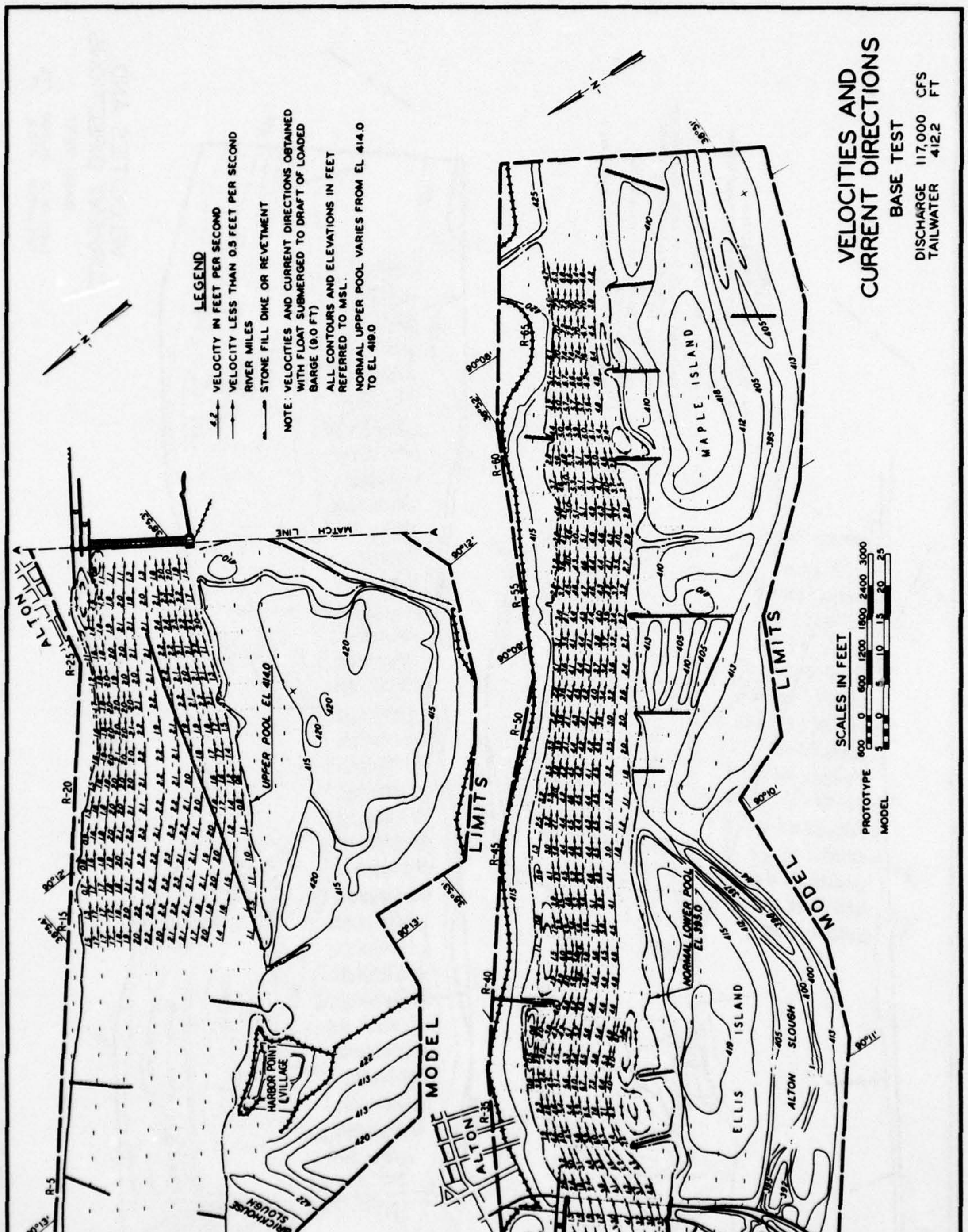
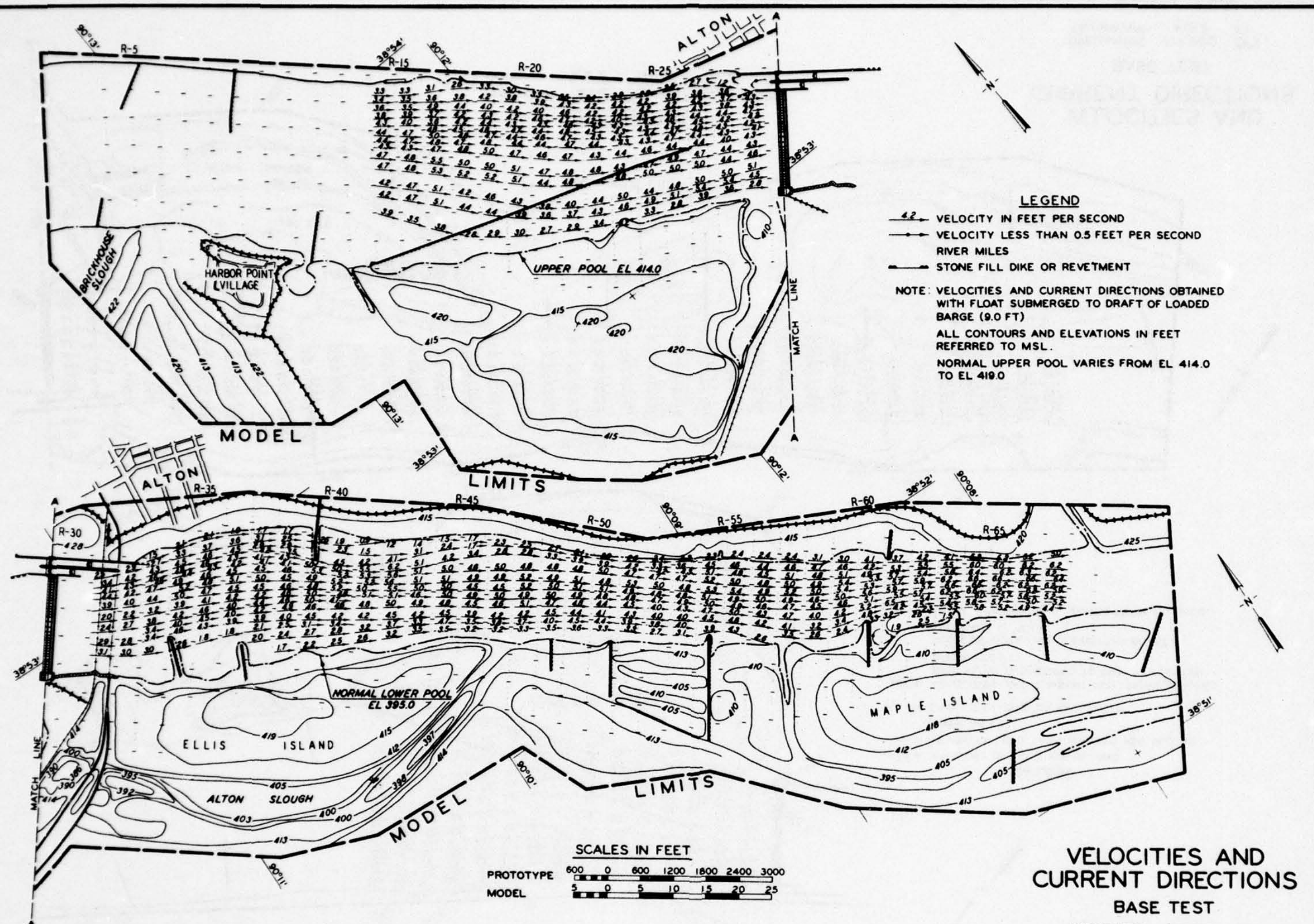

VELOCITIES AND
CURRENT DIRECTIONS
BASE TEST
DISCHARGE 117,000 CFS
TAILWATER 412.2 FT
LEGEND
VELOCITY IN FEET PER SECOND
VELOCITY LESS THAN 0.5 FEET PER SECOND
RIVER MILES
STONE FILL DIKE OR REVETMENT
NOTE: VELOCITIES AND CURRENT DIRECTIONS OBTAINED WITH FLOAT SUBMERGED TO DRAFT OF LOADED BARGE (9.0 FT)
ALL CONTOURS AND ELEVATIONS IN FEET REFERRED TO MSL.
NORMAL UPPER POOL VARIES FROM EL 414.0 TO EL 419.0
SCALES IN FEET
PROTOTYPE 600 0 600 1200 1800 2400 3000
MODEL 5 0 5 10 15 20 25
MODEL LIMITS
MATCH LINE
ALTON
HARBOR POINT VILLAGE
BRICKHOUSE SLOUGH
UPPER POOL EL 414.0
NORMAL LOWER POOL EL 393.0
MAPLE ISLAND
ELLIS ISLAND
ALTON SLOUGH

PLATE 1

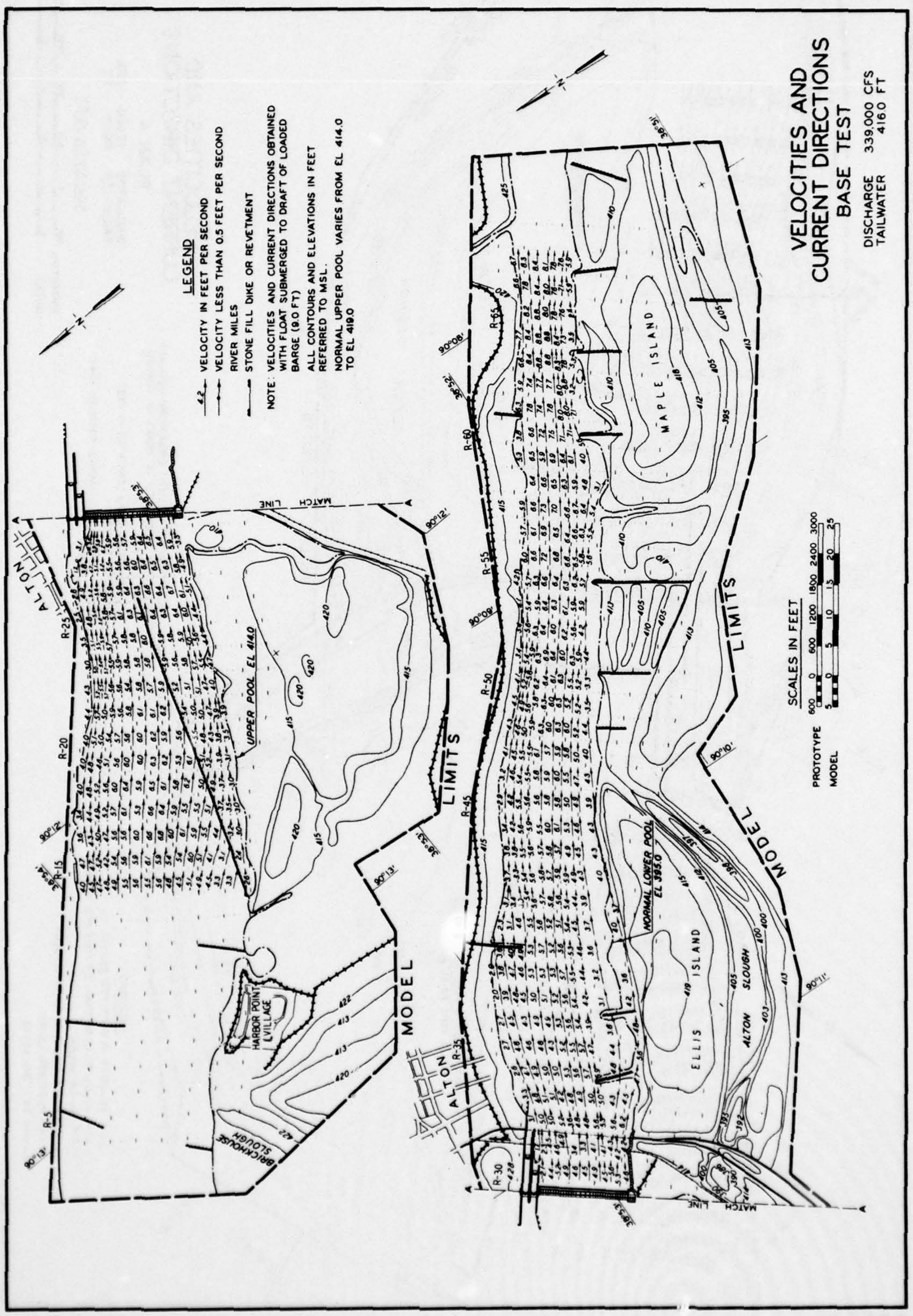


VELOCITIES AND
CURRENT DIRECTIONS

BASE TEST

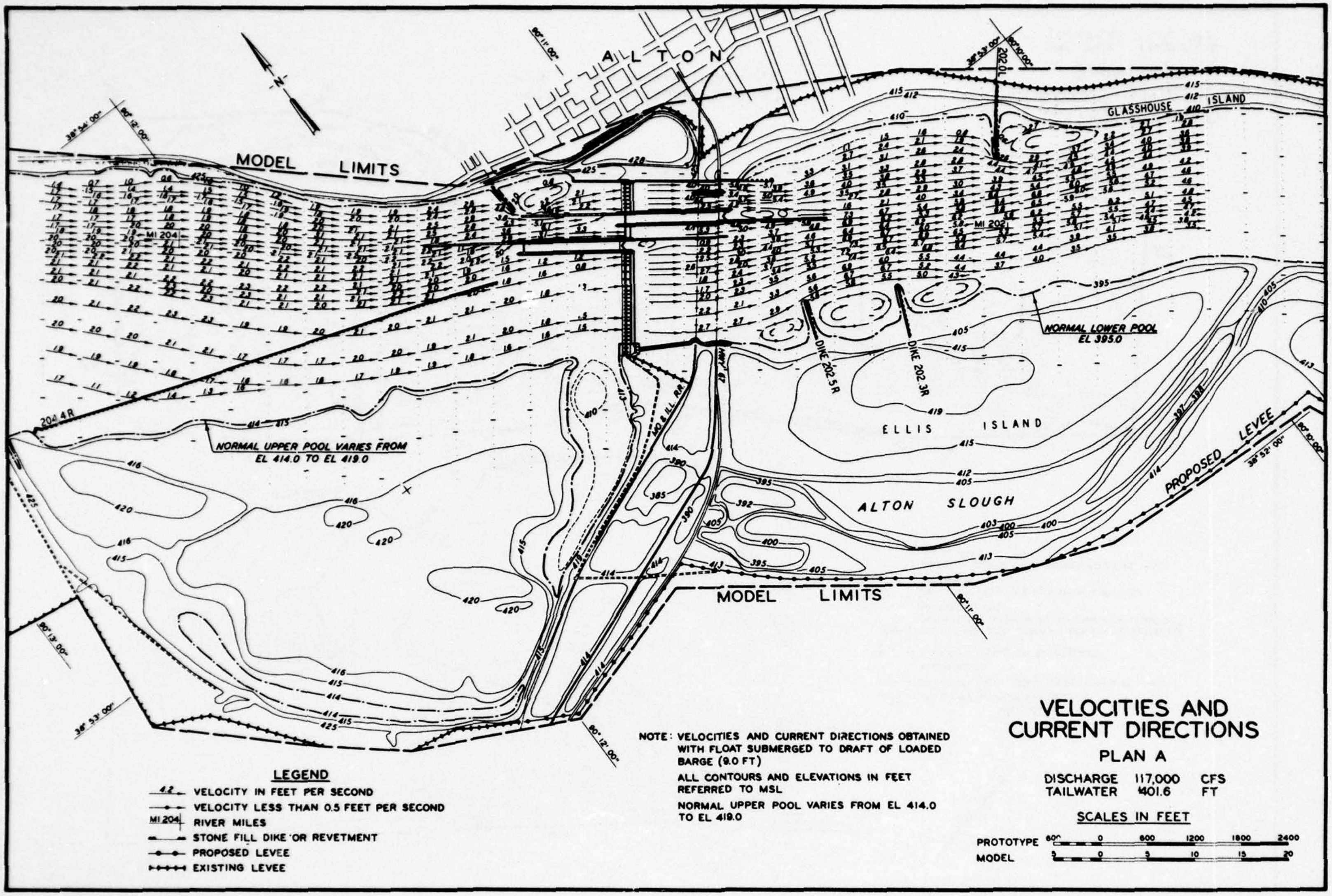
DISCHARGE 210,000 CFS
TAILWATER 412.2 FT

PLATE 2



VELOCITIES AND CURRENT DIRECTIONS
BASE TEST
DISCHARGE 339,000 CFS
TAILWATER 416.0 FT
LEGEND
4.2 VELOCITY IN FEET PER SECOND
VELOCITY LESS THAN 0.5 FEET PER SECOND
RIVER MILES
STONE FILL DIKE OR REVETMENT
NOTE: VELOCITIES AND CURRENT DIRECTIONS OBTAINED WITH FLOAT SUBMERGED TO DRAFT OF LOADED BARGE (9.0 FT)
ALL CONTOURS AND ELEVATIONS IN FEET REFERRED TO MSL.
NORMAL UPPER POOL VARIES FROM EL 414.0 TO EL 419.0
SCALES IN FEET
PROTOTYPE 600 0 600 1200 1800 2400 3000
MODEL 5 0 5 10 15 20 25
ALTON
HARBOR POINT VILLAGE
BRICKHOUSE SLOUGH
UPPER POOL EL 414.0
NORMAL LOWER POOL EL 395.0
MAPLE ISLAND
ELLIS ISLAND
ALTON SLOUGH
MODEL LIMITS
MATCH LINE

PLATE 3



VELOCITIES AND CURRENT DIRECTIONS

PLAN A

DISCHARGE 117,000 CFS
TAILWATER 401.6 FT

NOTE: VELOCITIES AND CURRENT DIRECTIONS OBTAINED WITH FLOAT SUBMERGED TO DRAFT OF LOADED BARGE (9.0 FT)

ALL CONTOURS AND ELEVATIONS IN FEET REFERRED TO MSL

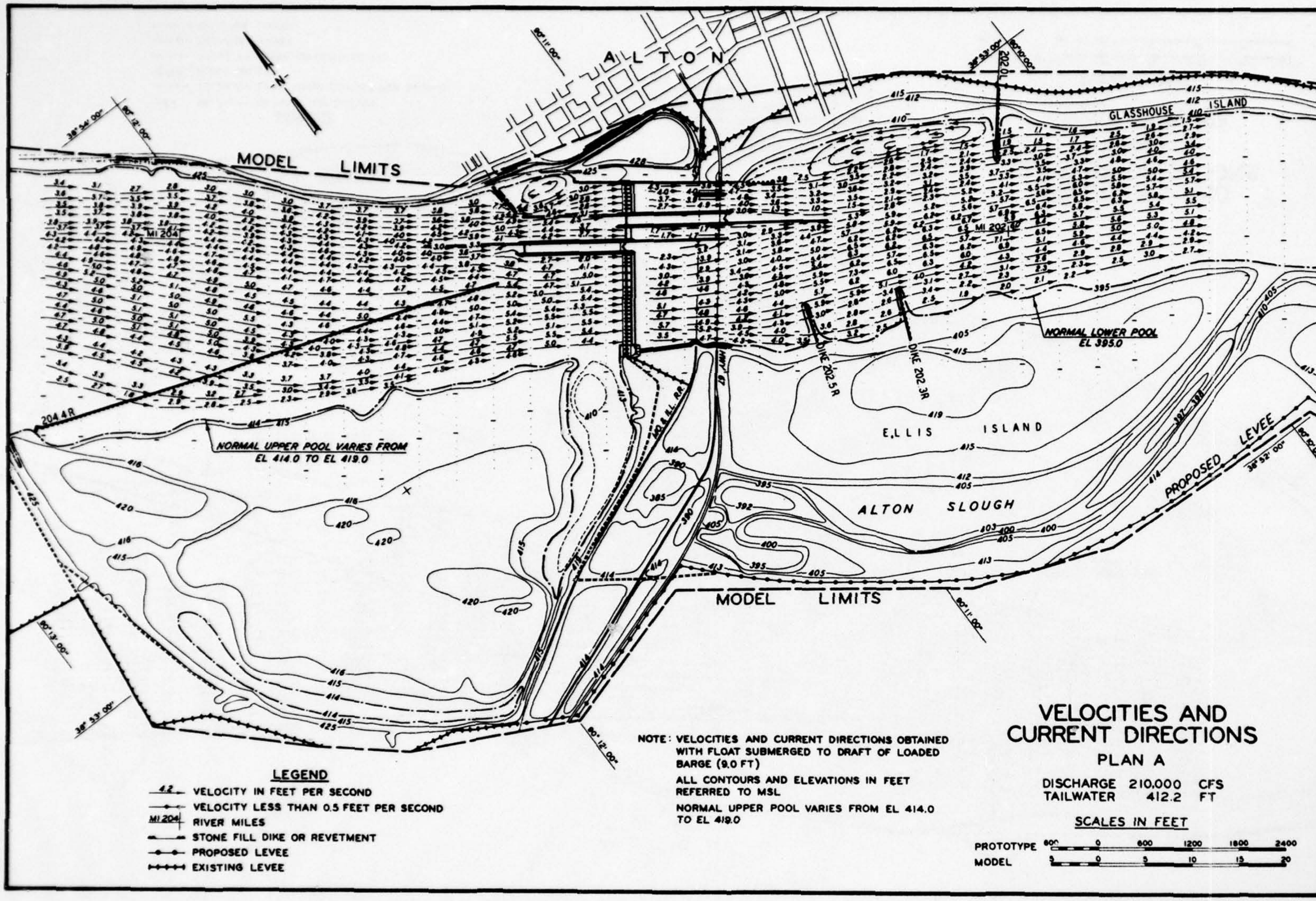
NORMAL UPPER POOL VARIES FROM EL 414.0 TO EL 419.0

LEGEND

- 4.2 VELOCITY IN FEET PER SECOND
- VELOCITY LESS THAN 0.5 FEET PER SECOND
- MI 204 RIVER MILES
- STONE FILL DIKE OR REVETMENT
- PROPOSED LEVEE
- EXISTING LEVEE

SCALES IN FEET

PROTOTYPE 600 0 600 1200 1800 2400
MODEL 5 0 5 10 15 20


VELOCITIES AND CURRENT DIRECTIONS
PLAN A
DISCHARGE 210,000 CFS
TAILWATER 412.2 FT
SCALES IN FEET
PROTOTYPE 600 0 600 1200 1800 2400
MODEL 5 0 5 10 15 20
NOTE: VELOCITIES AND CURRENT DIRECTIONS OBTAINED WITH FLOAT SUBMERGED TO DRAFT OF LOADED BARGE (9.0 FT)
ALL CONTOURS AND ELEVATIONS IN FEET REFERRED TO MSL
NORMAL UPPER POOL VARIES FROM EL 414.0 TO EL 419.0
LEGEND
4.2 VELOCITY IN FEET PER SECOND
VELOCITY LESS THAN 0.5 FEET PER SECOND
MI 204 RIVER MILES
STONE FILL DIKE OR REVETMENT
PROPOSED LEVEE
EXISTING LEVEE
ALTON
MODEL LIMITS
MODEL LIMITS
GLASSHOUSE ISLAND
ELLIS ISLAND
ALTON SLOUGH
PROPOSED LEVEE
NORMAL LOWER POOL EL 395.0
NORMAL UPPER POOL VARIES FROM EL 414.0 TO EL 419.0
DIKE 202.5R
DIKE 202.3R
202.0L
204.4R
MI 204
MI 202
HWY 67
N

AD-A077 529 ARMY ENGINEER WATERWAYS EXPERIMENT STATION VICKSBURG--ETC F/G 13/13
NAVIGATION CONDITIONS AT LOCKS AND DAM 26, MISSISSIPPI RIVER; H--ETC(U)
OCT 79 L J SHOWS , J J FRANCO
UNCLASSIFIED WES-HL-79-19 NL

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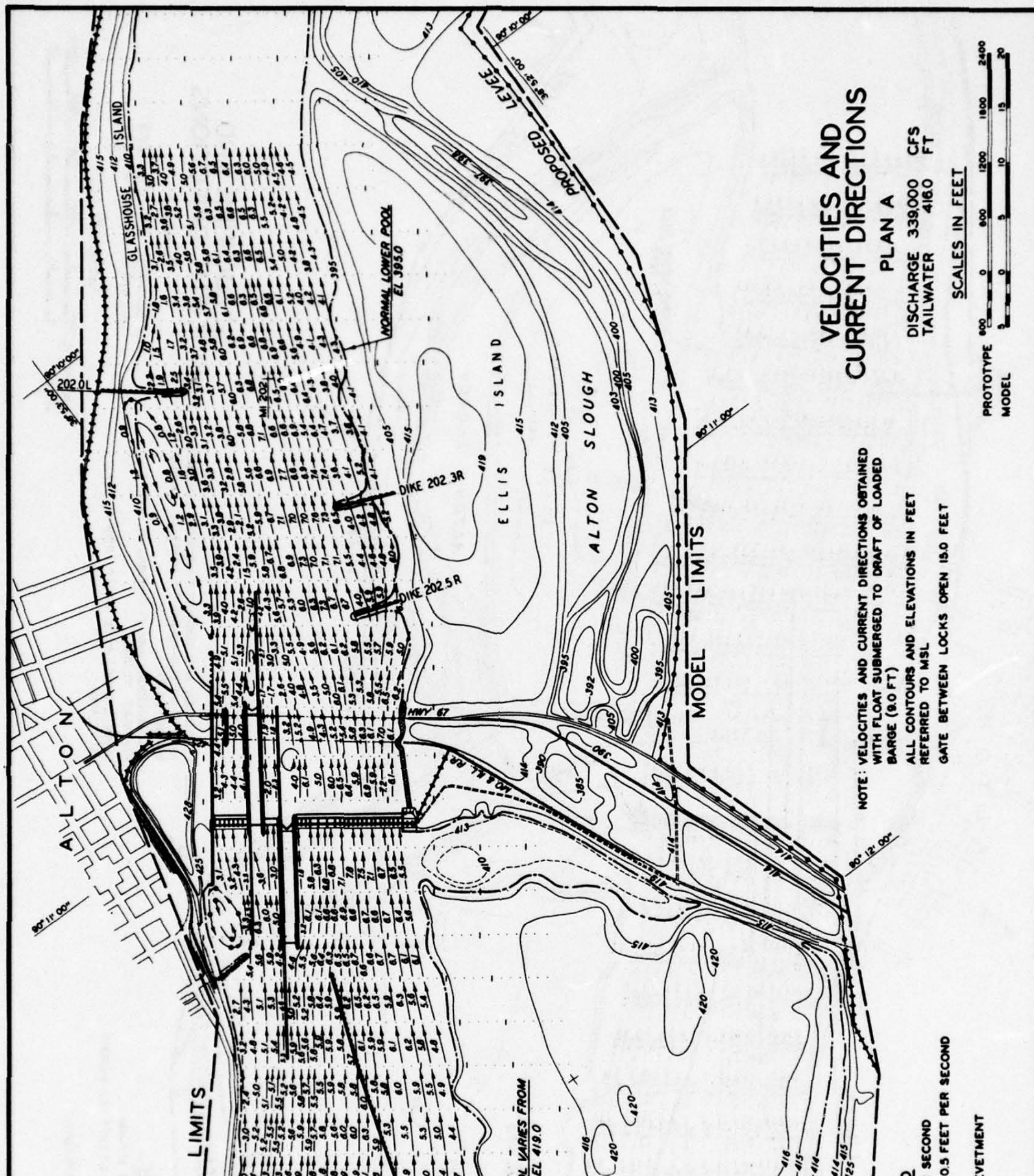
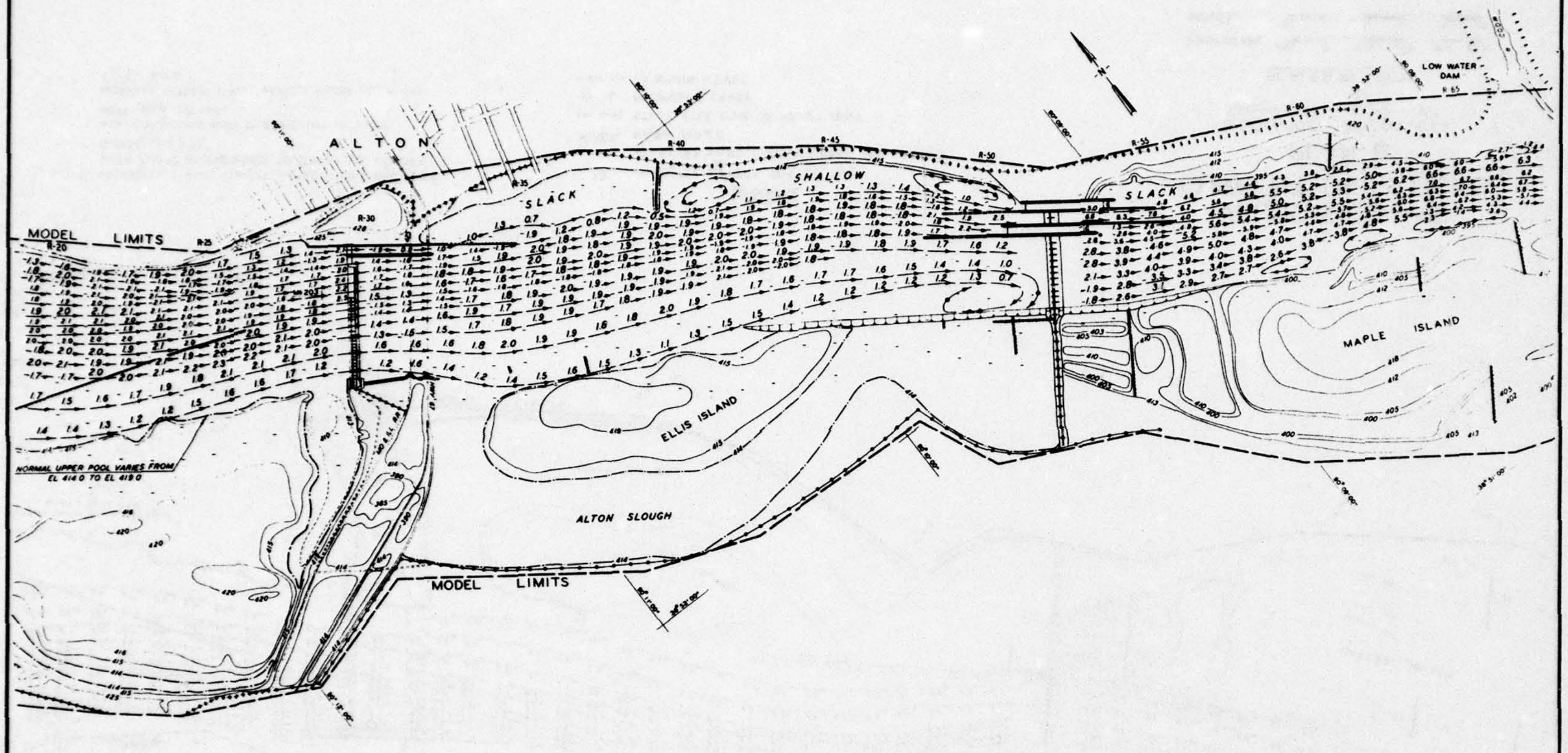

VELOCITIES AND CURRENT DIRECTIONS
PLAN A
DISCHARGE 339,000 CFS
TAILWATER 416.0 FT
SCALES IN FEET
PROTOTYPE
MODEL
NOTE: VELOCITIES AND CURRENT DIRECTIONS OBTAINED WITH FLOAT SUBMERGED TO DRAFT OF LOADED BARGE (9.0 FT)
ALL CONTOURS AND ELEVATIONS IN FEET REFERRED TO MSL
GATE BETWEEN LOCKS OPEN 15.0 FEET
LEGEND
VELOCITY IN FEET PER SECOND
VELOCITY LESS THAN 0.5 FEET PER SECOND
RIVER MILES
STONE FILL DIKE OR REVETMENT
PROPOSED LEVEE
EXISTING LEVEE
ALTON
MODEL LIMITS
GLASSHOUSE ISLAND
ELLIS ISLAND
ALTON SLOUGH
PROPOSED LEVEE
NORMAL LOWER POOL EL 395.0
NORMAL UPPER POOL VARIES FROM EL 414.0 TO EL 419.0
DIKE 202.3R
DIKE 202.5R
HWY 67

PLATE 6



NOTE: VELOCITIES AND CURRENT DIRECTIONS OBTAINED WITH FLOAT SUBMERGED TO DRAFT OF LOADED BARGE (9.0 FT)

ALL CONTOURS AND ELEVATIONS IN FEET REFERRED TO MSL

NORMAL UPPER POOL VARIES FROM EL 414.0 TO EL 419.0

LEGEND

4.2 VELOCITY IN FEET PER SECOND

VELOCITY LESS THAN 0.5 FEET PER SECOND

MI 204 RIVER MILES

STONE FILL DIKE OR REVETMENT

PROPOSED LEVEE

EXISTING LEVEE

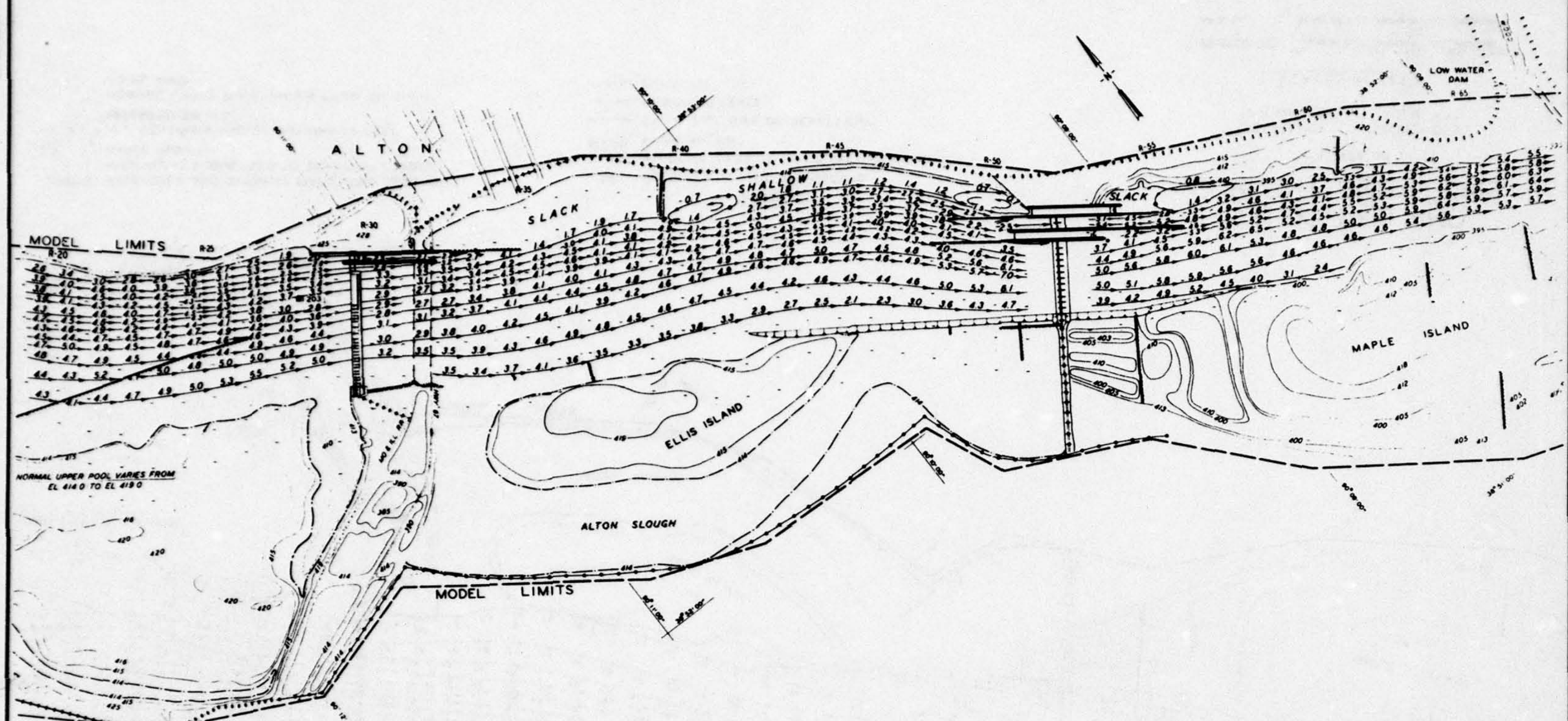
VELOCITIES AND CURRENT DIRECTIONS

PLAN B

DISCHARGE 117,000 CFS
TAILWATER 401.6 FT

SCALES IN FEET

PROTOTYPE 600 0 600 1200 1800 2400
MODEL 5 0 5 10 15 20



NOTE: VELOCITIES AND CURRENT DIRECTIONS OBTAINED WITH FLOAT SUBMERGED TO DRAFT OF LOADED BARGE (9.0 FT)

ALL CONTOURS AND ELEVATIONS IN FEET REFERRED TO MSL

NORMAL UPPER POOL VARIES FROM EL 414.0 TO EL 419.0

LEGEND

- 4.2 VELOCITY IN FEET PER SECOND
- VELOCITY LESS THAN 0.5 FEET PER SECOND
- MI 204 RIVER MILES
- STONE FILL DIKE OR REVETMENT
- PROPOSED LEVEE
- EXISTING LEVEE

VELOCITIES AND CURRENT DIRECTIONS

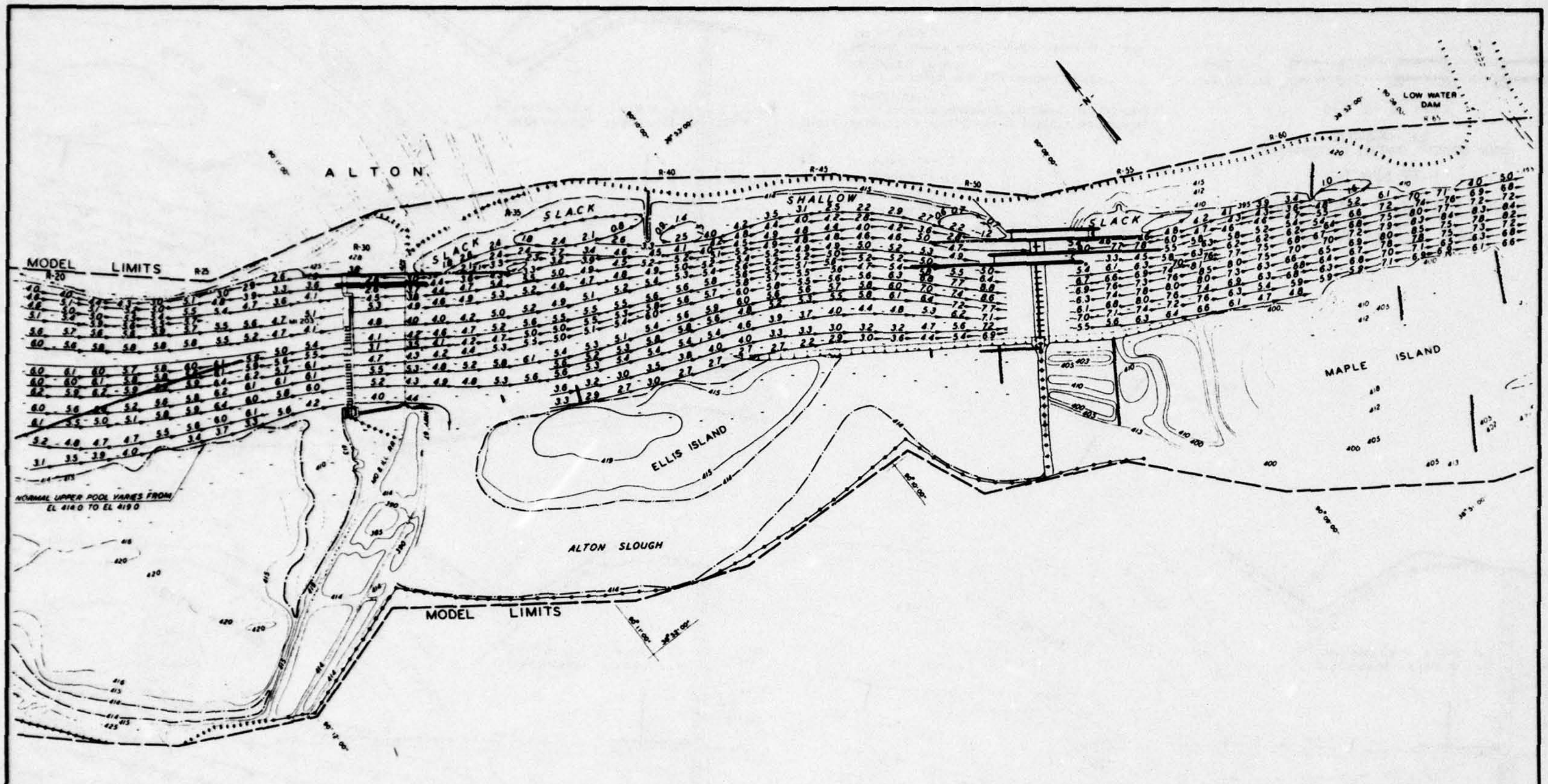
PLAN B

DISCHARGE 210,000 CFS
TAILWATER 412.2 FT

SCALES IN FEET

PROTOTYPE 600 0 600 1200 1800 2400
MODEL 5 0 5 10 15 20

PLATE 8



NOTE: VELOCITIES AND CURRENT DIRECTIONS OBTAINED WITH FLOAT SUBMERGED TO DRAFT OF LOADED BARGE (9.0 FT)

ALL CONTOURS AND ELEVATIONS IN FEET REFERRED TO MSL

NORMAL UPPER POOL VARIES FROM EL 414.0 TO EL 419.0

LEGEND

4.2 VELOCITY IN FEET PER SECOND

VELOCITY LESS THAN 0.5 FEET PER SECOND

MI 204 RIVER MILES

STONE FILL DIKE OR REVETMENT

PROPOSED LEVEE

EXISTING LEVEE

VELOCITIES AND CURRENT DIRECTIONS

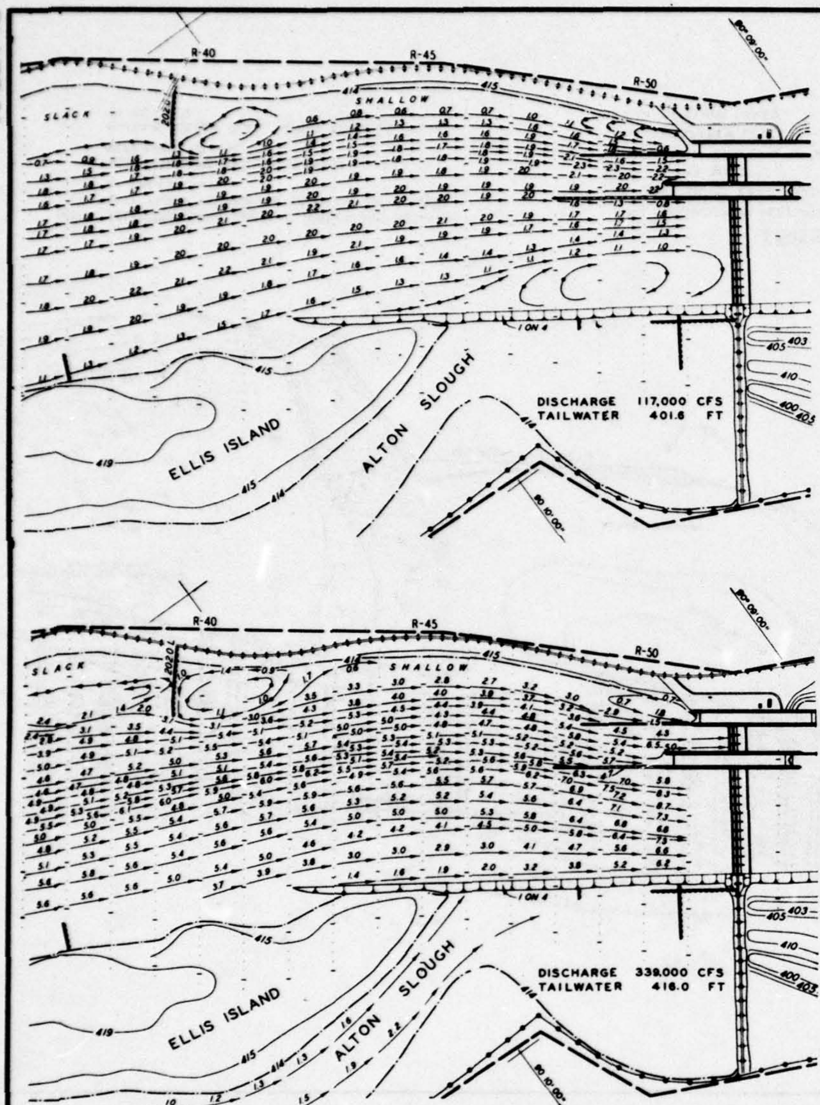
PLAN B

DISCHARGE 339,000 CFS
TAILWATER 416.0 FT

SCALES IN FEET

PROTOTYPE 600 0 600 1200 1800 2400
MODEL 5 0 5 10 15 20

PLATE 9



LEGEND

- VELOCITY IN FEET PER SECOND
- VELOCITY LESS THAN 0.5 FEET PER SECOND
- RIVER MILES
- STONE FILL DIKE OR REVETMENT
- PROPOSED LEVEE
- EXISTING LEVEE

NOTE: VELOCITIES AND CURRENT DIRECTIONS OBTAINED WITH FLOAT SUBMERGED TO DRAFT OF LOADED BARGE (9.0 FT)

ALL CONTOURS AND ELEVATIONS IN FEET REFERRED TO MSL

NORMAL UPPER POOL VARIES FROM EL 414.0 TO EL 419.0

VELOCITIES AND CURRENT DIRECTIONS

PLAN B-1

DISCHARGES 117,000, 210,000, AND 339,000 CFS

SCALES IN FEET

PROTOTYPE 600 0 600 1200 1800 2400

MODEL 5 0 5 10 15 20

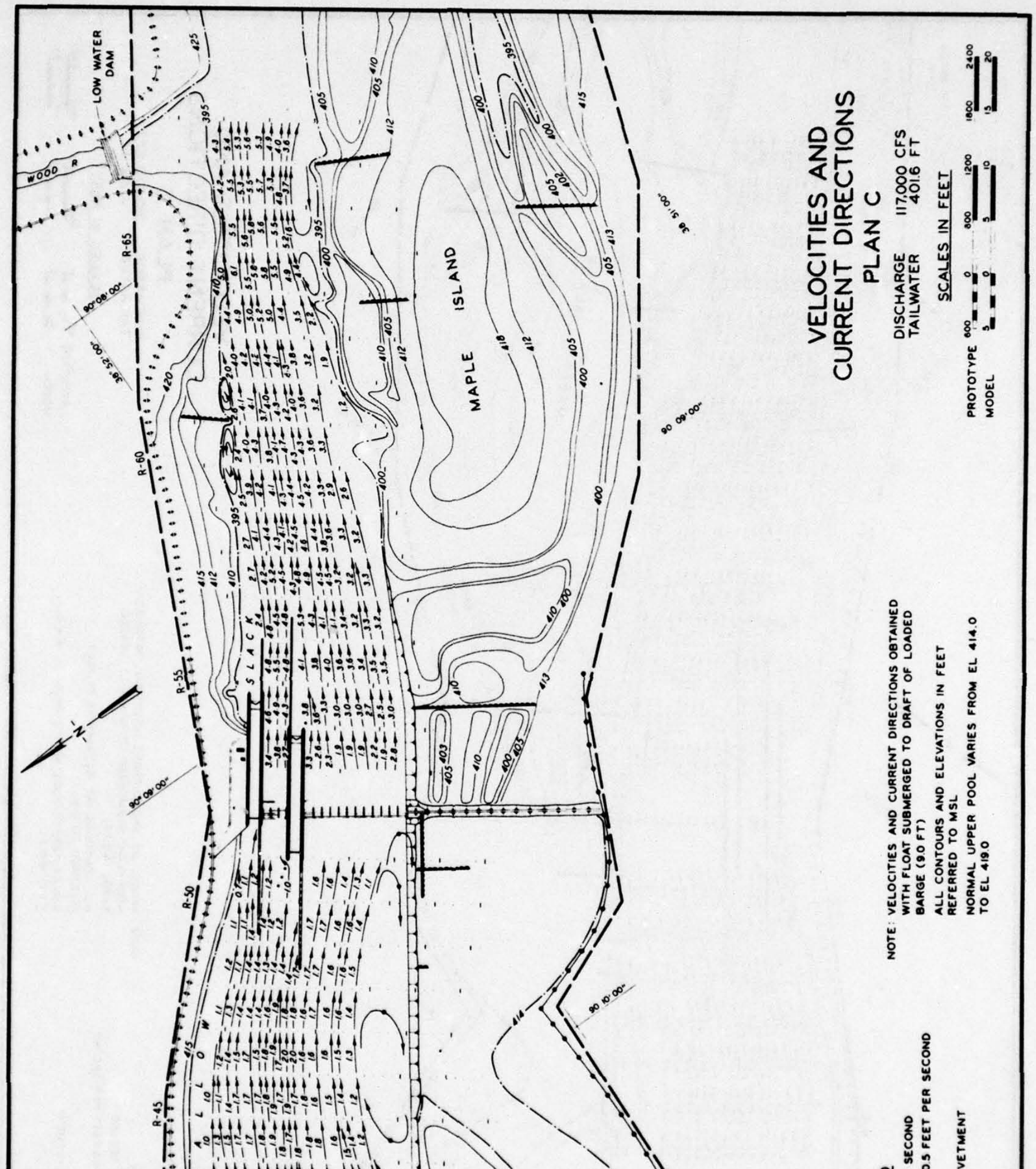

VELOCITIES AND
CURRENT DIRECTIONS
PLAN C
DISCHARGE 117,000 CFS
TAILWATER 401.6 FT
SCALES IN FEET
PROTOTYPE 600 0 600 1200 1800 2400
MODEL 5 0 5 10 15 20
NOTE: VELOCITIES AND CURRENT DIRECTIONS OBTAINED WITH FLOAT SUBMERGED TO DRAFT OF LOADED BARGE (9.0 FT)
ALL CONTOURS AND ELEVATIONS IN FEET REFERRED TO MSL
NORMAL UPPER POOL VARIES FROM EL 414.0 TO EL 419.0
LEGEND
4.2 VELOCITY IN FEET PER SECOND
VELOCITY LESS THAN 0.5 FEET PER SECOND
MI 204 RIVER MILES
STONE FILL DIKE OR REVETMENT
PROPOSED LEVEE
EXISTING LEVEE
LOW WATER DAM
WOOD R
MAPLE ISLAND
ELLIS ISLAND
ALTON SLOUGH
SLACK
SHALLOW
R-40
R-45
R-50
R-55
R-60
R-65

PLATE 11

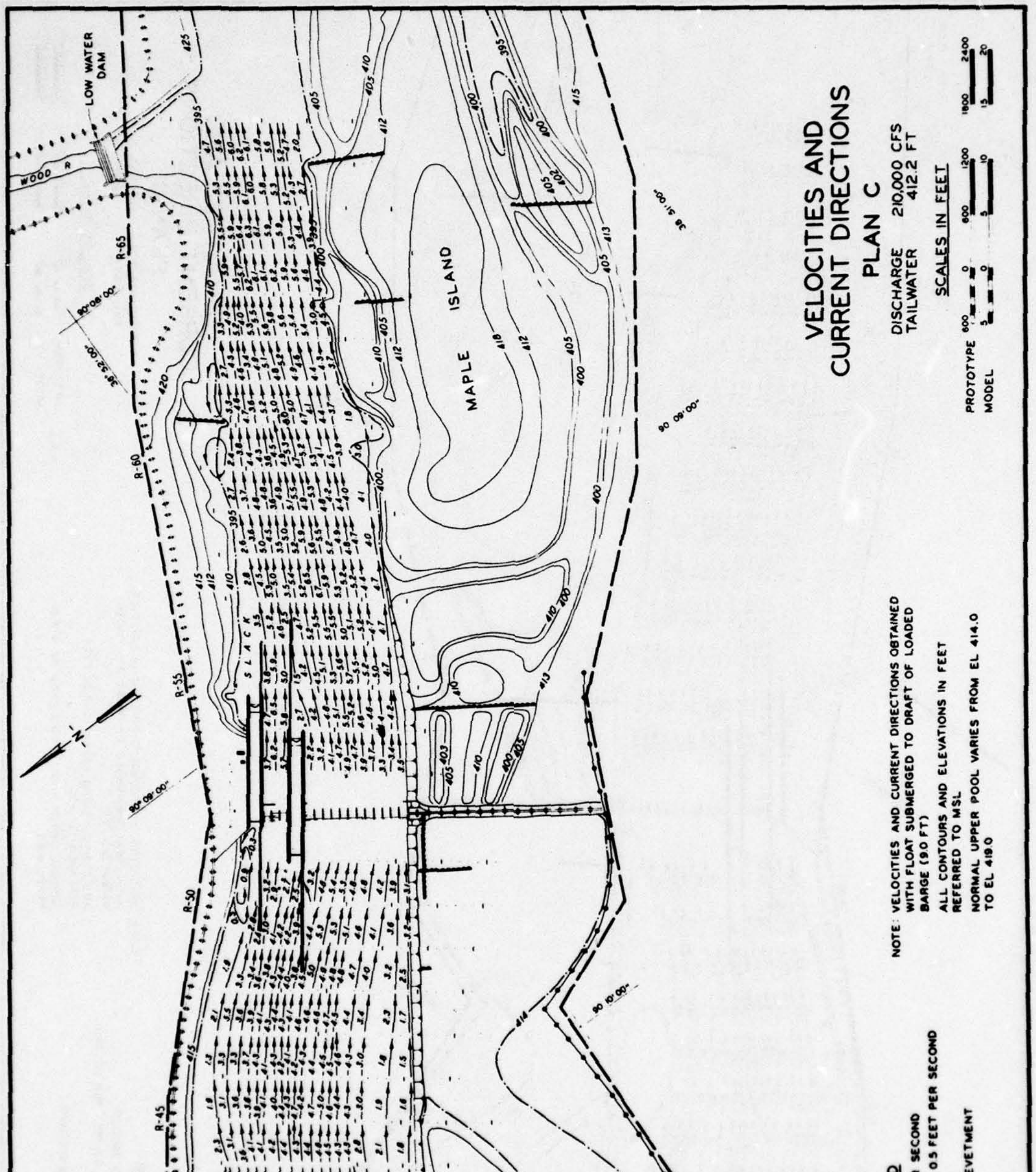

VELOCITIES AND
CURRENT DIRECTIONS
PLAN C
DISCHARGE 210,000 CFS
TAILWATER 412.2 FT
SCALES IN FEET
PROTOTYPE
MODEL
NOTE: VELOCITIES AND CURRENT DIRECTIONS OBTAINED WITH FLOAT SUBMERGED TO DRAFT OF LOADED BARGE (9.0 FT)
ALL CONTOURS AND ELEVATIONS IN FEET REFERRED TO MSL
NORMAL UPPER POOL VARIES FROM EL 414.0 TO EL 419.0
LEGEND
VELOCITY IN FEET PER SECOND
VELOCITY LESS THAN 0.5 FEET PER SECOND
RIVER MILES
STONE FILL DIKE OR REVETMENT
PROPOSED LEVEE
EXISTING LEVEE
MAPLE ISLAND
ELLIS ISLAND
ALTON SLOUGH
SLACK
LOW WATER DAM
WOOD R
R-40
R-45
R-50
R-55
R-60
R-65

PLATE 12

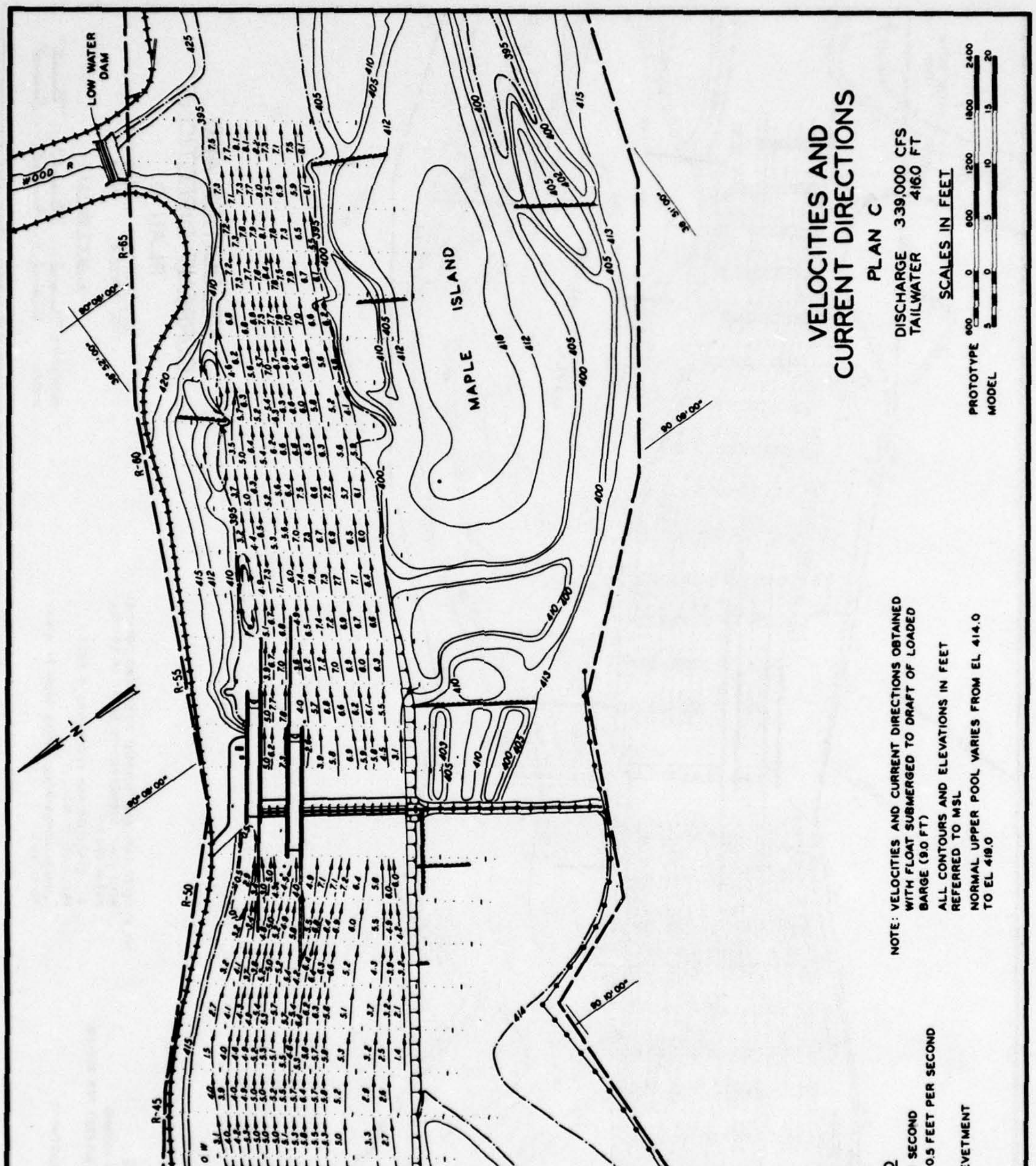
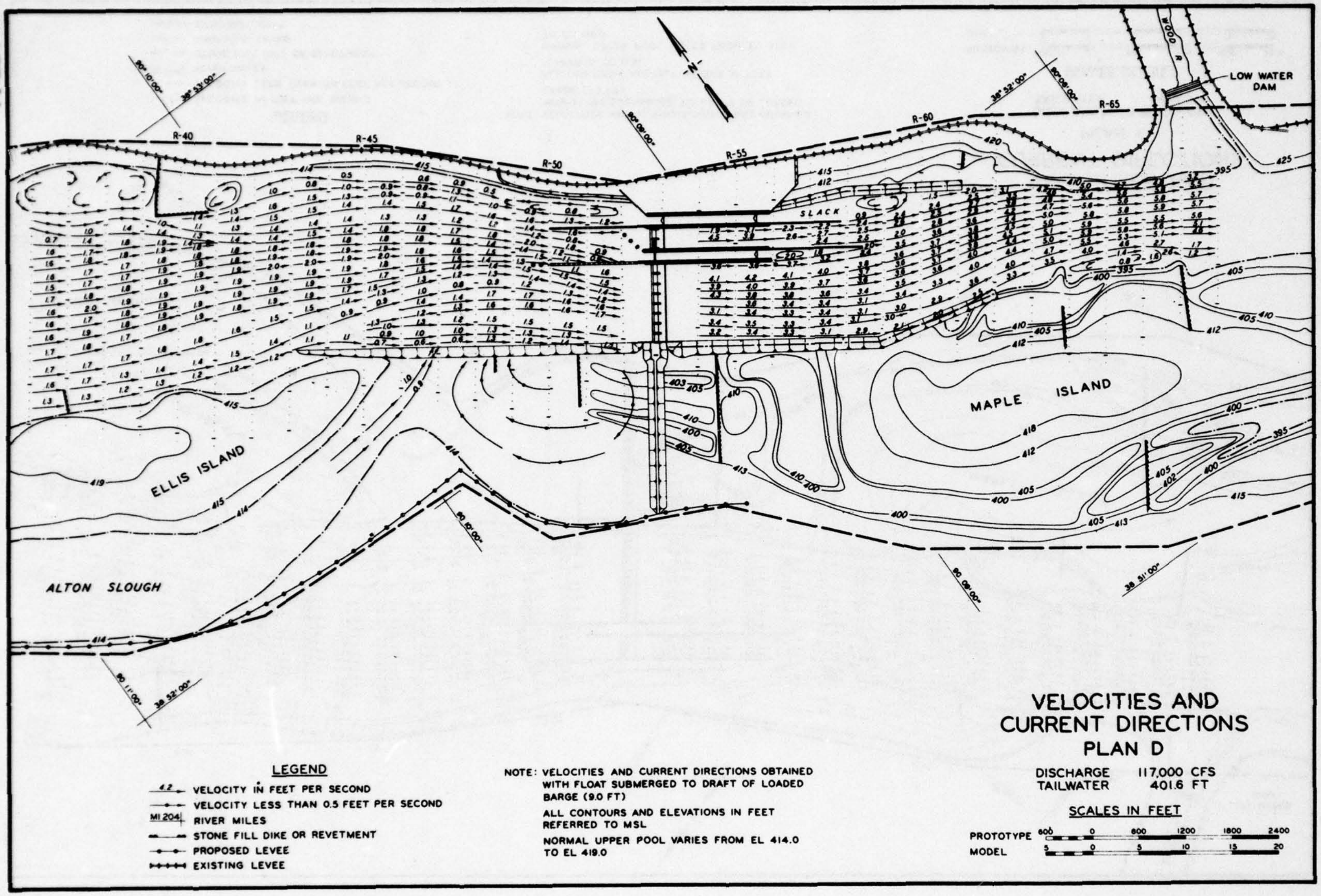

VELOCITIES AND
CURRENT DIRECTIONS
PLAN C
DISCHARGE 339,000 CFS
TAILWATER 416.0 FT
SCALES IN FEET
PROTOTYPE
MODEL
NOTE: VELOCITIES AND CURRENT DIRECTIONS OBTAINED WITH FLOAT SUBMERGED TO DRAFT OF LOADED BARGE (9.0 FT)
ALL CONTOURS AND ELEVATIONS IN FEET REFERRED TO MSL
NORMAL UPPER POOL VARIES FROM EL 414.0 TO EL 419.0
LEGEND
VELOCITY IN FEET PER SECOND
VELOCITY LESS THAN 0.5 FEET PER SECOND
RIVER MILES
STONE FILL DIKE OR REVETMENT
PROPOSED LEVEE
EXISTING LEVEE
LOW WATER DAM
WOOD R
MAPLE ISLAND
ELLIS ISLAND
ALTON SLOUGH
SHALLOW
R-40
R-45
R-50
R-55
R-60
R-65

PLATE 13

PLATE 14



VELOCITIES AND CURRENT DIRECTIONS
PLAN D

DISCHARGE 117,000 CFS
TAILWATER 401.6 FT

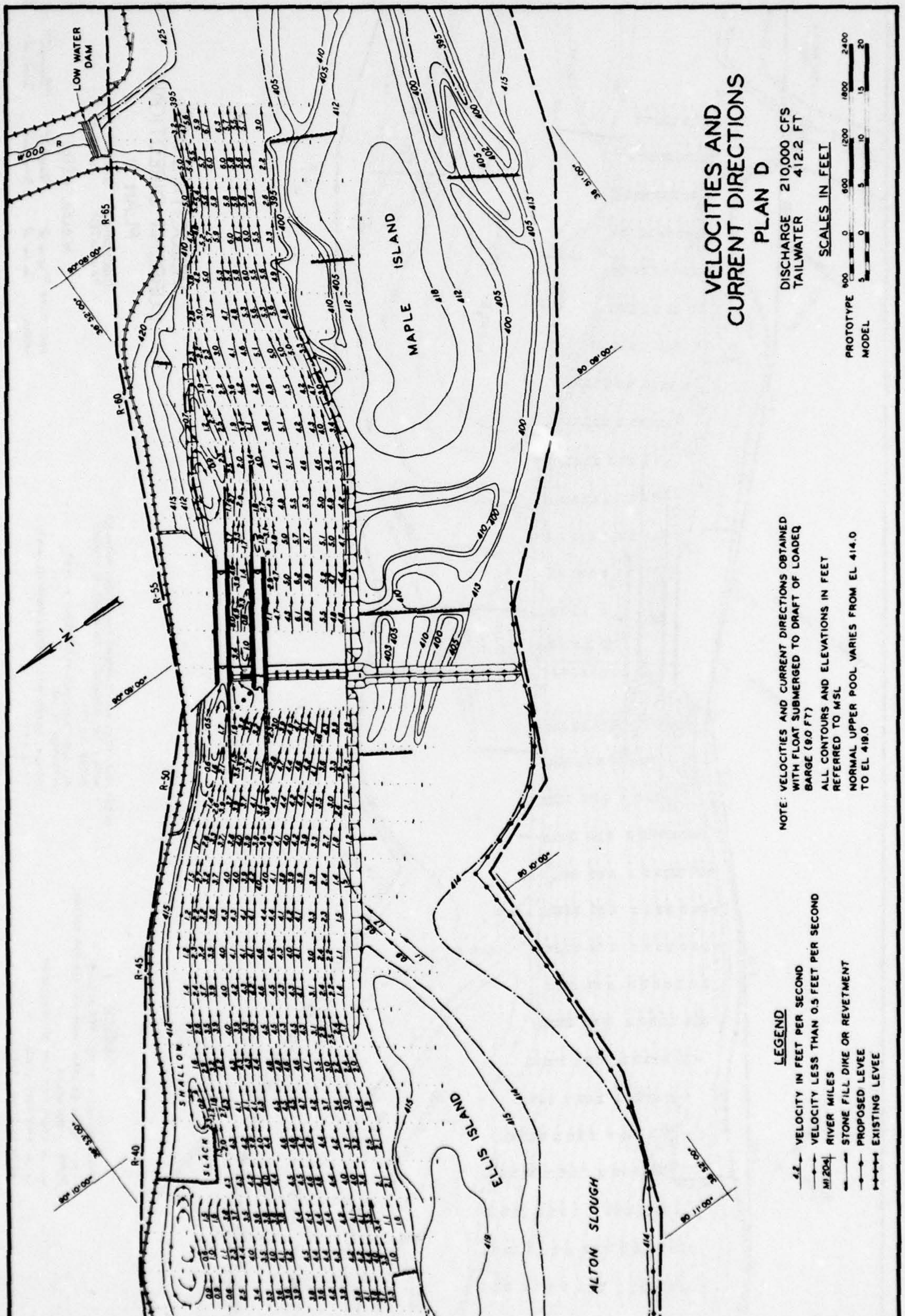
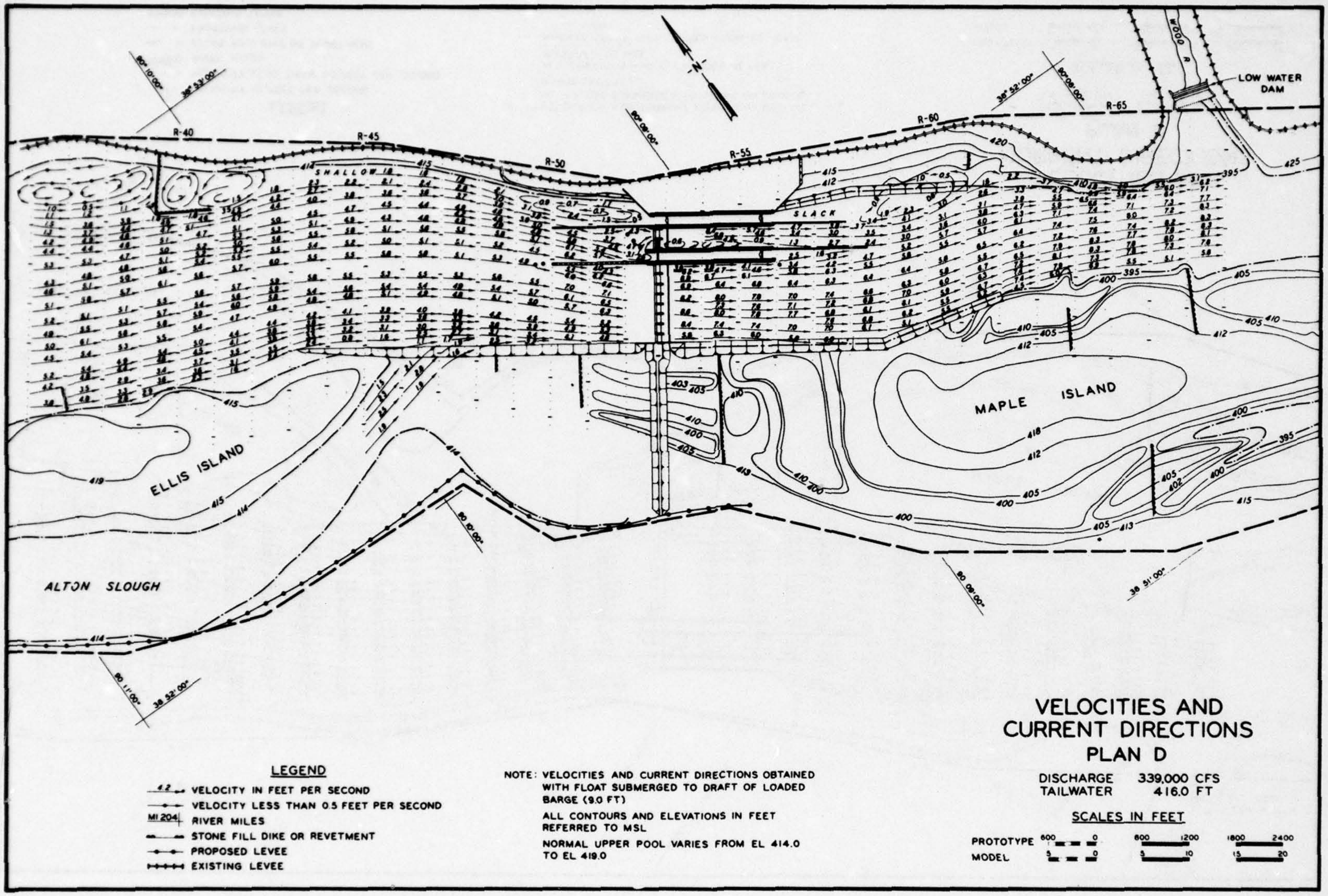
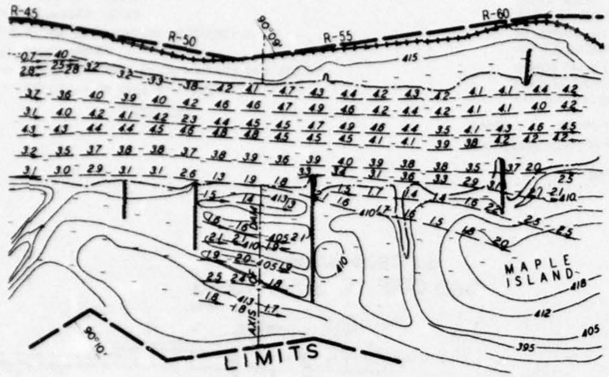
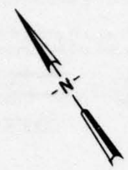

VELOCITIES AND CURRENT DIRECTIONS
PLAN D
DISCHARGE 210,000 CFS
TAILWATER 412.2 FT
SCALES IN FEET
PROTOTYPE
MODEL
LEGEND
4.2 VELOCITY IN FEET PER SECOND
VELOCITY LESS THAN 0.5 FEET PER SECOND
MI 204 RIVER MILES
STONE FILL DIKE OR REVETMENT
PROPOSED LEVEE
EXISTING LEVEE
NOTE: VELOCITIES AND CURRENT DIRECTIONS OBTAINED WITH FLOAT SUBMERGED TO DRAFT OF LOADED BARGE (9.0 FT)
ALL CONTOURS AND ELEVATIONS IN FEET REFERRED TO MSL
NORMAL UPPER POOL VARIES FROM EL 414.0 TO EL 419.0
LOW WATER DAM
WOOD R
MAPLE ISLAND
ELLIS ISLAND
ALTON SLOUGH
SHALLOW
SLACK
R-40
R-45
R-50
R-55
R-60
R-65

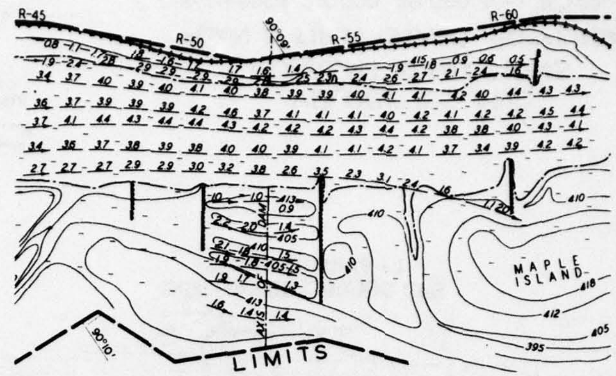
PLATE 15


VELOCITIES AND
CURRENT DIRECTIONS
PLAN D
DISCHARGE 339,000 CFS
TAILWATER 416.0 FT
SCALES IN FEET
PROTOTYPE
MODEL
LEGEND
VELOCITY IN FEET PER SECOND
VELOCITY LESS THAN 0.5 FEET PER SECOND
RIVER MILES
STONE FILL DIKE OR REVETMENT
PROPOSED LEVEE
EXISTING LEVEE
NOTE: VELOCITIES AND CURRENT DIRECTIONS OBTAINED WITH FLOAT SUBMERGED TO DRAFT OF LOADED BARGE (9.0 FT)
ALL CONTOURS AND ELEVATIONS IN FEET REFERRED TO MSL
NORMAL UPPER POOL VARIES FROM EL 414.0 TO EL 419.0
LOW WATER DAM
WOOD R
MAPLE ISLAND
ELLIS ISLAND
ALTON SLOUGH
SHALLOW
SLACK
R-40
R-45
R-50
R-55
R-60
R-65
N

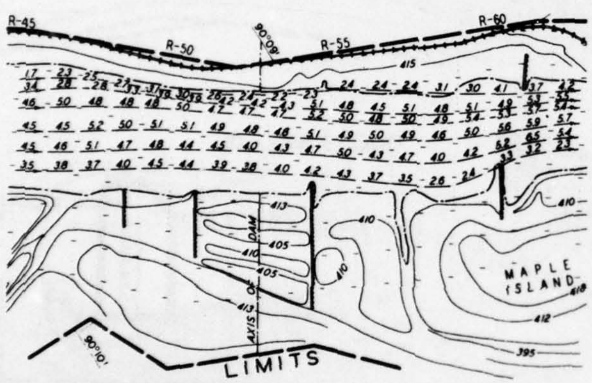


DISCHARGE 360,000 CFS
TAILWATER 427.8 FT





DISCHARGE 315,000 CFS
TAILWATER 425.2 FT



DISCHARGE 210,000 CFS
TAILWATER 413.0 FT

LEGEND

- 4.2 VELOCITY IN FEET PER SECOND
- VELOCITY LESS THAN 0.5 FEET PER SECOND
- MI 204 RIVER MILES
- STONE FILL DIKE OR REVETMENT
- 6.2 AVERAGE SPOT VELOCITY
- 5.8 BOTTOM SPOT VELOCITY

NOTE: VELOCITIES AND CURRENT DIRECTIONS OBTAINED WITH FLOAT SUBMERGED TO DRAFT OF LOADED BARGE (9.0 FT)

ALL CONTOURS AND ELEVATIONS IN FEET REFERRED TO MSL.

NORMAL UPPER POOL VARIES FROM EL 414.0 TO EL 419.0

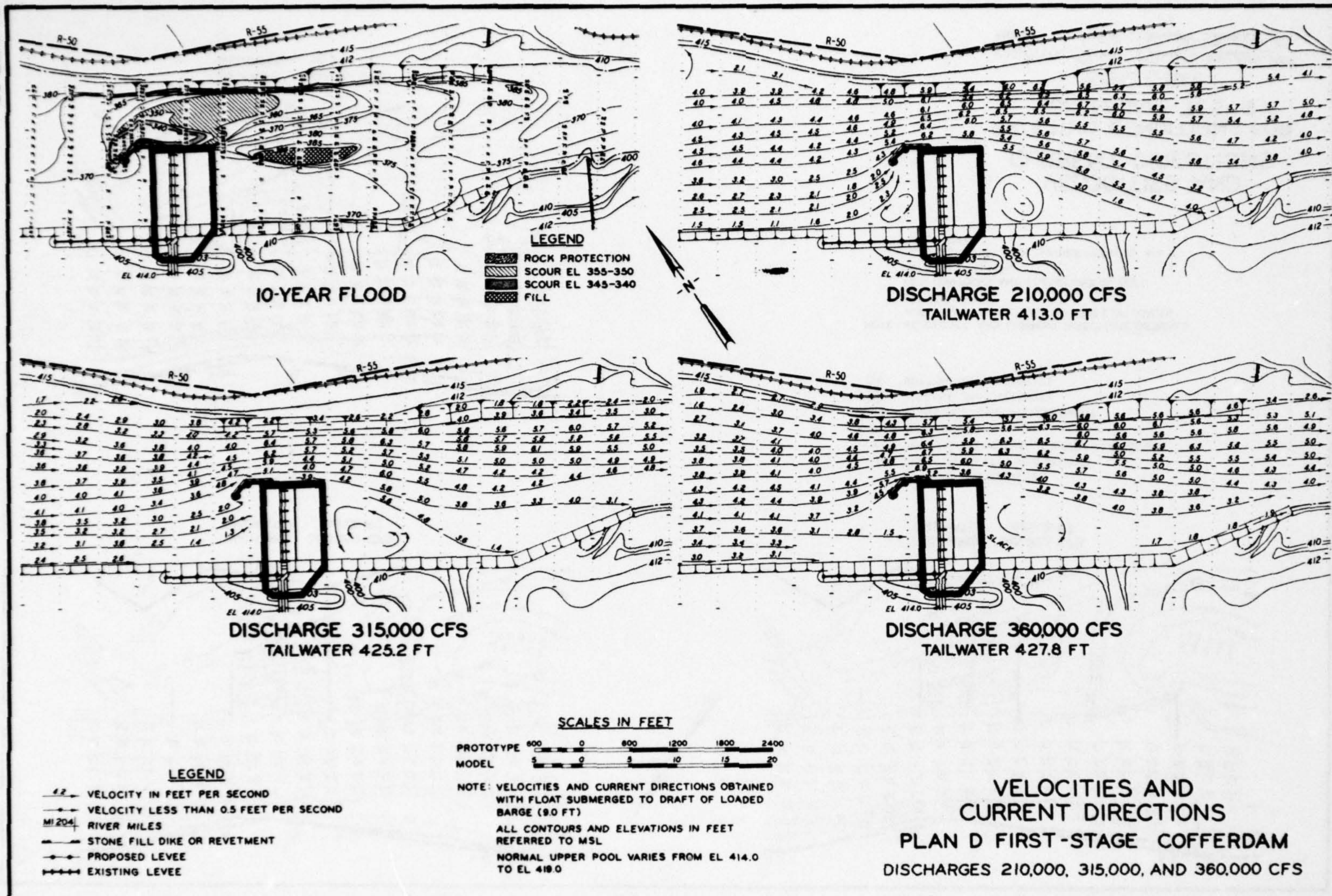
VELOCITIES AND CURRENT DIRECTIONS

EXISTING CONDITIONS FOR COFFERDAM TESTS

SCALES IN FEET

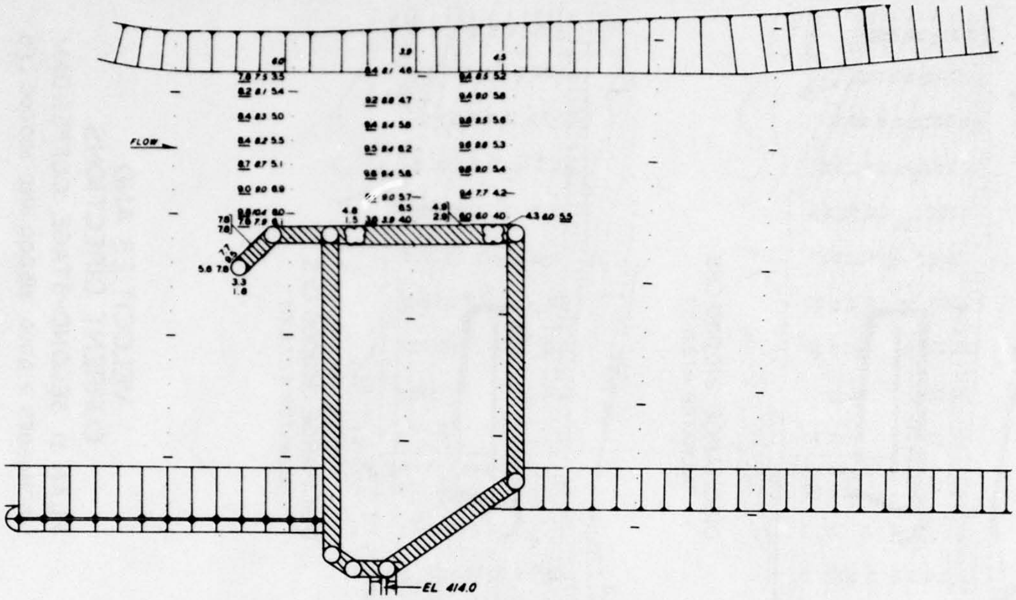
PROTOTYPE 600 0 600 1200 1800 2400 3000

MODEL 5 0 5 10 15 20 25

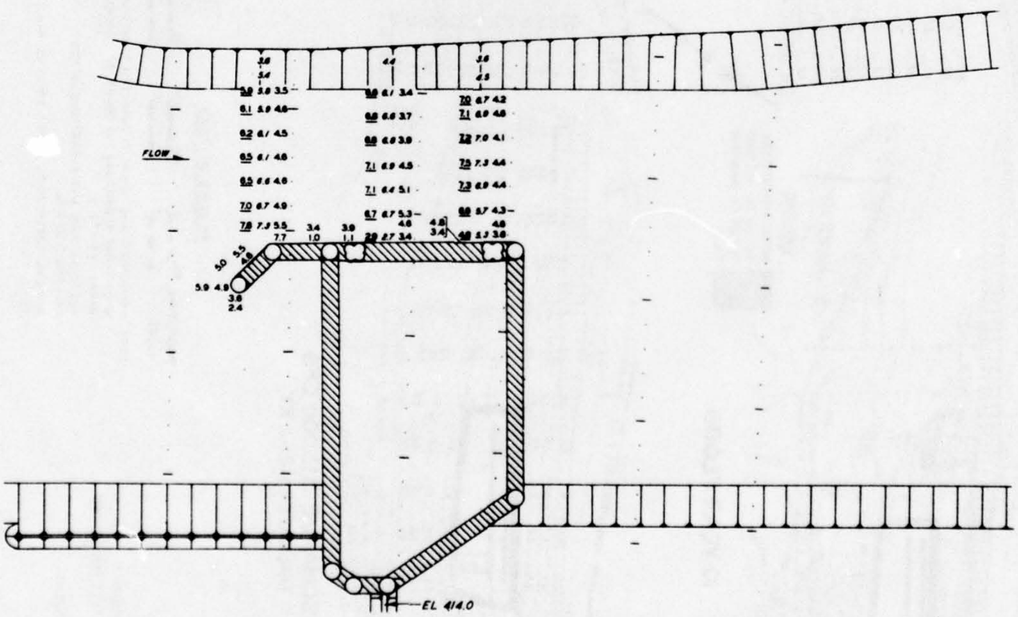


VELOCITIES AND CURRENT DIRECTIONS
PLAN D FIRST-STAGE COFFERDAM
DISCHARGES 210,000, 315,000, AND 360,000 CFS

PLATE 18



DISCHARGE 240,000 CFS
TAILWATER EL 412.8 FT



DISCHARGE 360,000 CFS
TAILWATER EL 427.8 FT

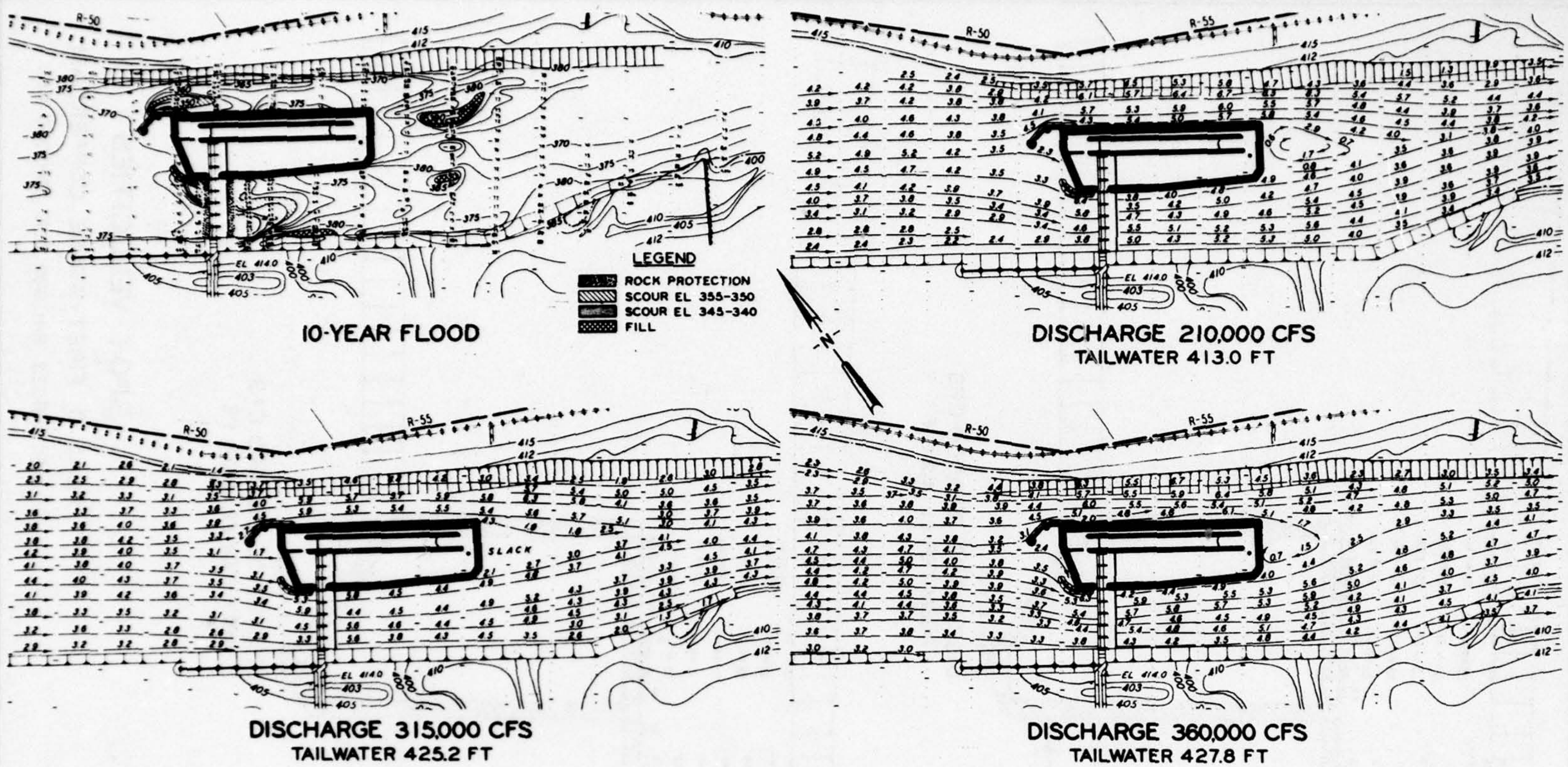
LEGEND

4.0 VELOCITY AT 20% OF DEPTH
3.4 VELOCITY AT 60% OF DEPTH
2.7 BOTTOM VELOCITY

SPOT VELOCITIES

PLAN D FIRST-STAGE COFFERDAM

DISCHARGES 240,000 AND 360,000 CFS



SCALES IN FEET

PROTOTYPE 800 0 800 1200 1800 2400
MODEL 5 0 5 10 15 20

LEGEND

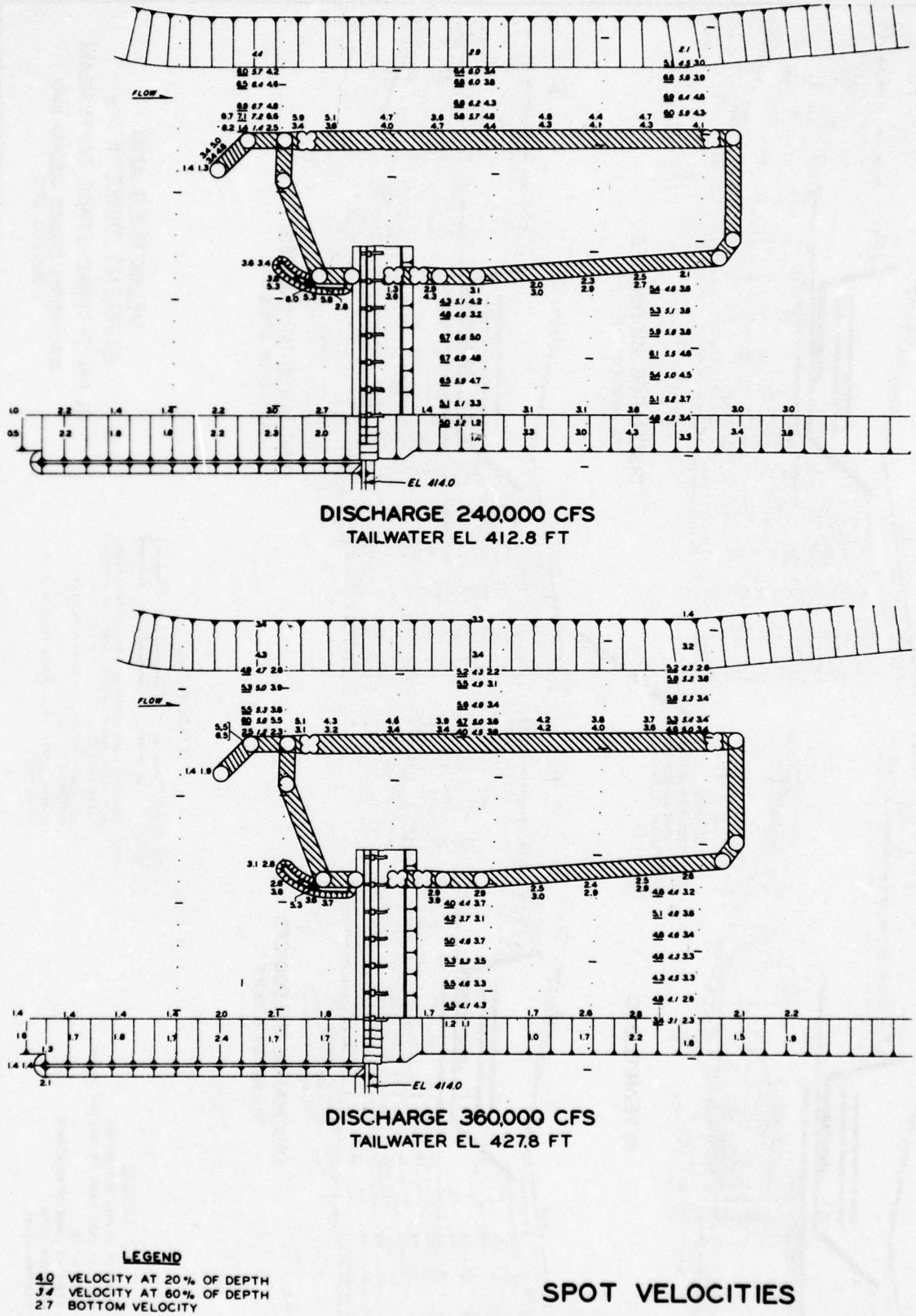
4.2 — VELOCITY IN FEET PER SECOND
• — VELOCITY LESS THAN 0.5 FEET PER SECOND
MI 204 RIVER MILES
— — STONE FILL DIKE OR REVETMENT
• • PROPOSED LEVEE
• • • • • EXISTING LEVEE

NOTE: VELOCITIES AND CURRENT DIRECTIONS OBTAINED WITH FLOAT SUBMERGED TO DRAFT OF LOADED BARGE (9.0 FT)

ALL CONTOURS AND ELEVATIONS IN FEET REFERRED TO MSL

NORMAL UPPER POOL VARIES FROM EL 414.0 TO EL 419.0

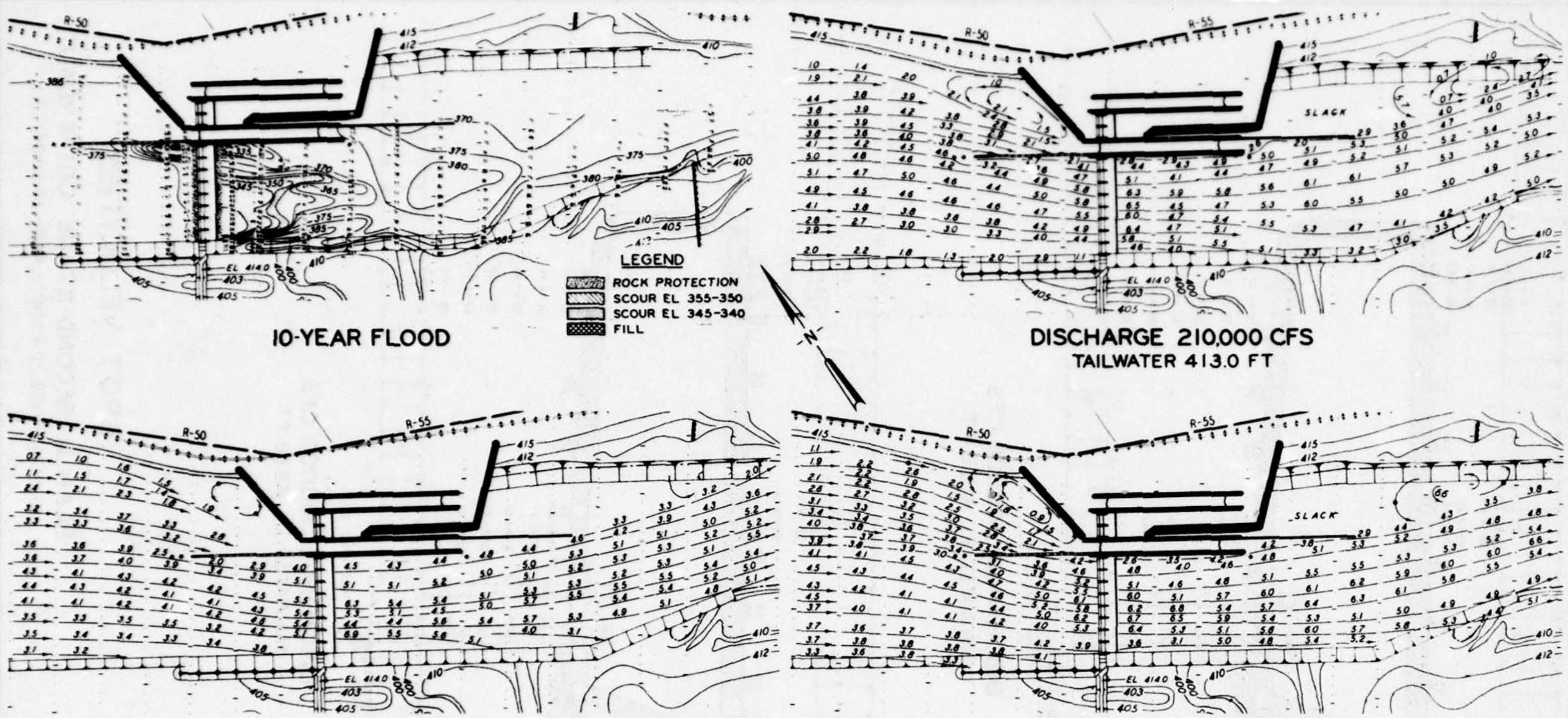
VELOCITIES AND
CURRENT DIRECTIONS
PLAN D SECOND-STAGE COFFERDAM
DISCHARGES 210,000, 315,000, AND, 360,000 CFS



SPOT VELOCITIES

PLAN D SECOND-STAGE COFFERDAM

DISCHARGES 240,000 AND 360,000 CFS



10-YEAR FLOOD

DISCHARGE 210,000 CFS
TAILWATER 413.0 FT

DISCHARGE 315,000 CFS
TAILWATER 425.2 FT

DISCHARGE 360,000 CFS
TAILWATER 427.8 FT

LEGEND

- 4.2 VELOCITY IN FEET PER SECOND
- VELOCITY LESS THAN 0.5 FEET PER SECOND
- MI 204 RIVER MILES
- STONE FILL DIKE OR REVETMENT
- PROPOSED LEVEE
- EXISTING LEVEE

SCALES IN FEET

PROTOTYPE 600 0 600 1200 1800 2400
MODEL 5 0 5 10 15 20

NOTE: VELOCITIES AND CURRENT DIRECTIONS OBTAINED WITH FLOAT SUBMERGED TO DRAFT OF LOADED BARGE (9.0 FT)

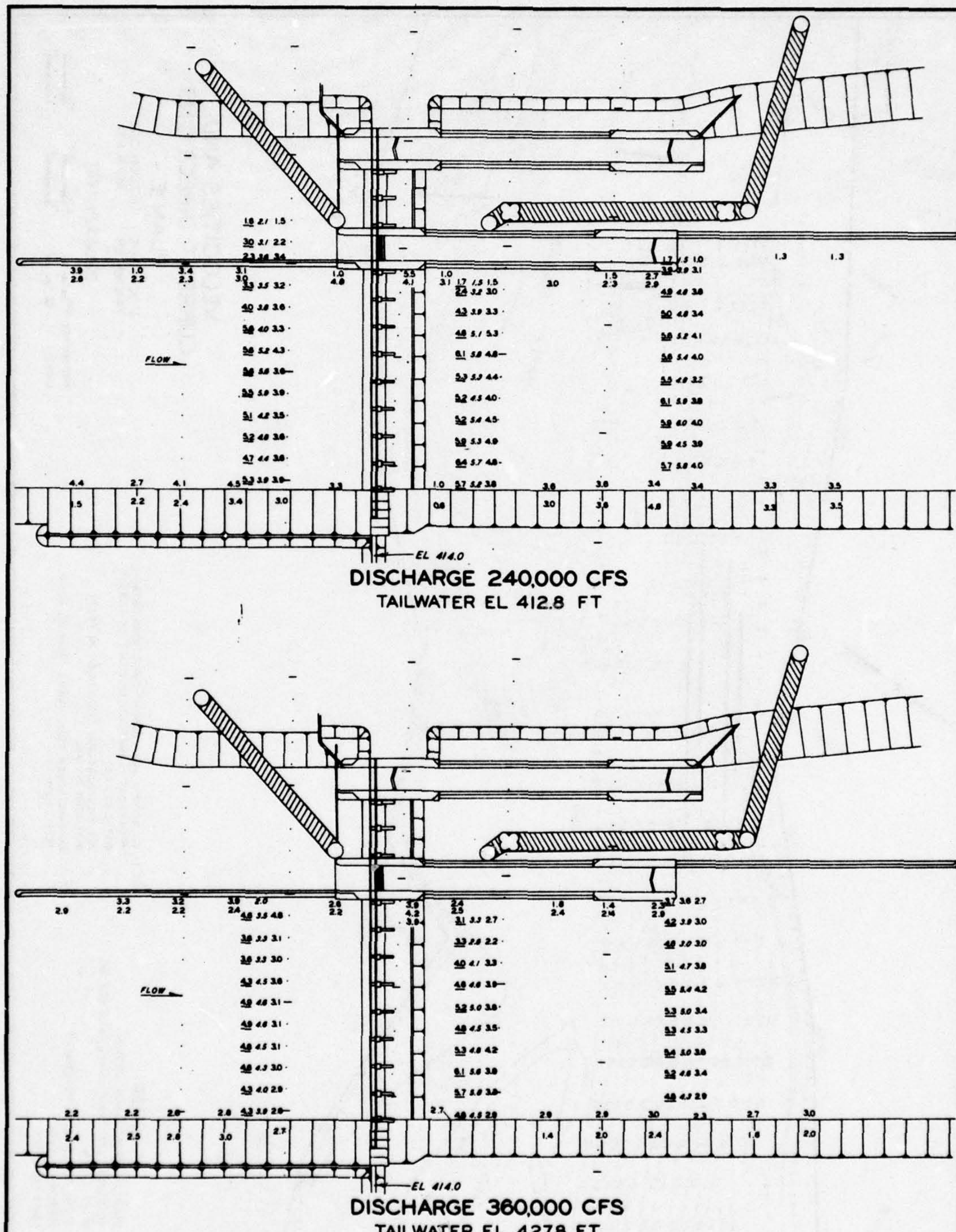
ALL CONTOURS AND ELEVATIONS IN FEET REFERRED TO MSL

NORMAL UPPER POOL VARIES FROM EL 414.0 TO EL 419.0

VELOCITIES AND CURRENT DIRECTIONS

PLAN D THIRD-STAGE COFFERDAM

DISCHARGES 210,000, 315,000, AND 360,000 CFS



LEGEND

4.0 VELOCITY AT 20% OF DEPTH
3.4 VELOCITY AT 60% OF DEPTH
2.7 BOTTOM VELOCITY

SPOT VELOCITIES
PLAN D THIRD-STAGE COFFERDAM
DISCHARGES 240,000 AND 360,000 CFS

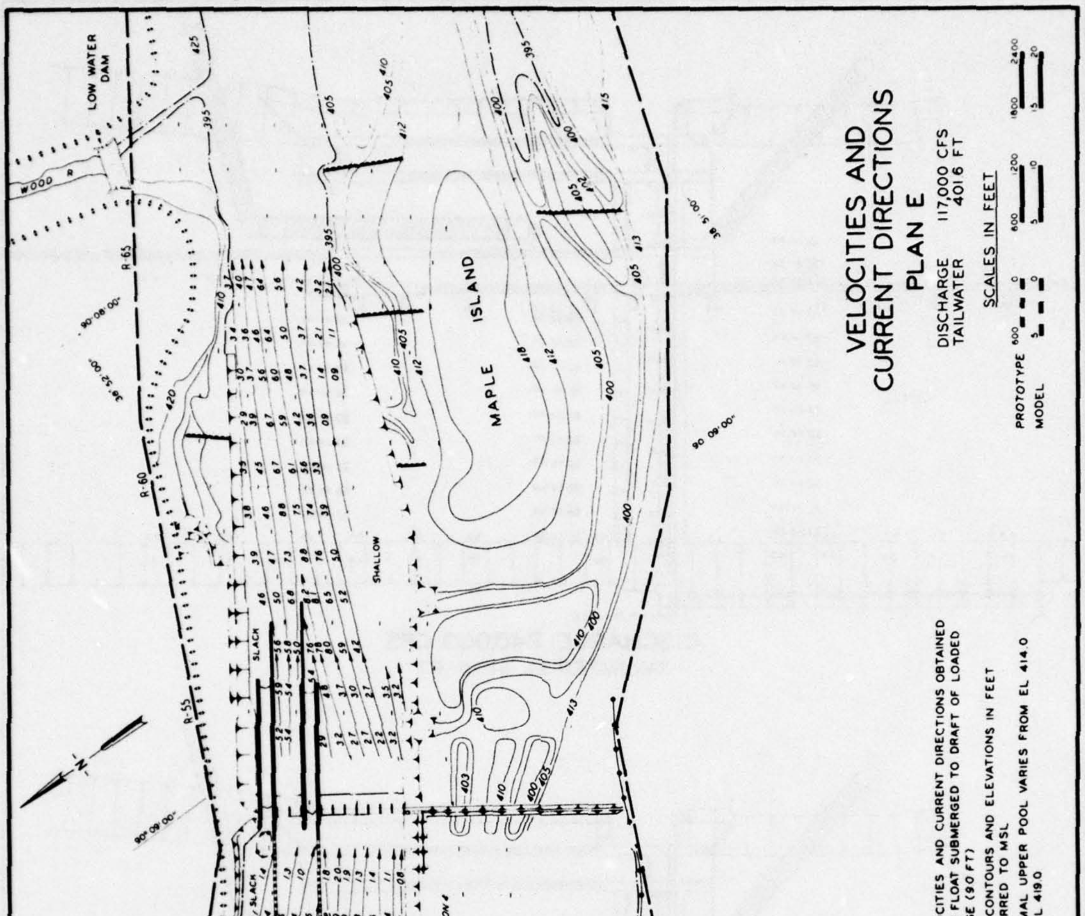
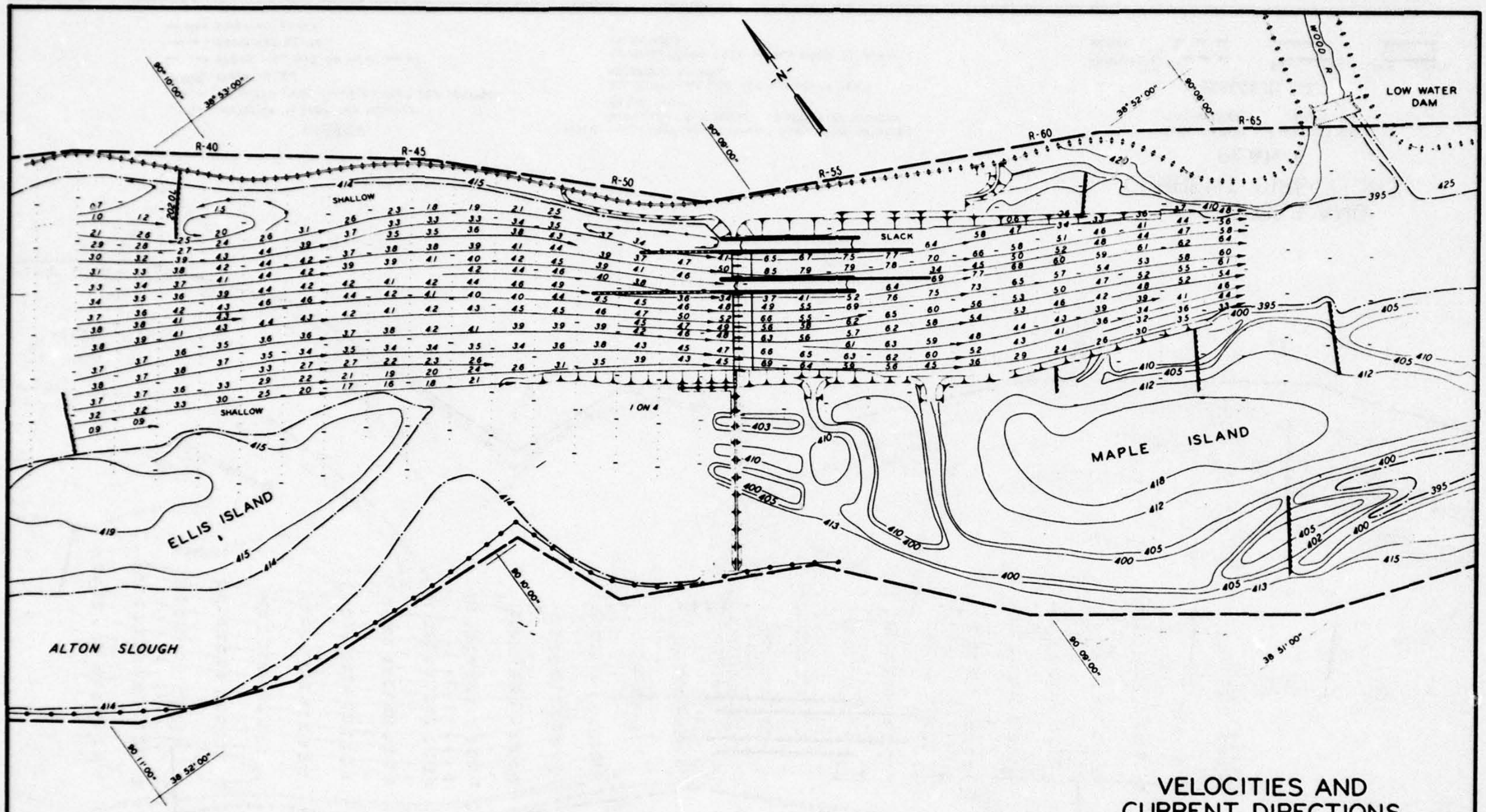

VELOCITIES AND CURRENT DIRECTIONS
PLAN E
DISCHARGE 117,000 CFS
TAILWATER 401.6 FT
SCALES IN FEET
PROTOTYPE
MODEL
NOTE: VELOCITIES AND CURRENT DIRECTIONS OBTAINED WITH FLOAT SUBMERGED TO DRAFT OF LOADED BARGE (9.0 FT)
ALL CONTOURS AND ELEVATIONS IN FEET REFERRED TO MSL
NORMAL UPPER POOL VARIES FROM EL 414.0 TO EL 419.0
LEGEND
VELOCITY IN FEET PER SECOND
VELOCITY LESS THAN 0.5 FEET PER SECOND
RIVER MILES
STONE FILL DIKE OR REVETMENT
PROPOSED LEVEE
EXISTING LEVEE
LOW WATER DAM
WOOD R
MAPLE ISLAND
ELLIS ISLAND
ALTON SLOUGH
SHALLOW
SLACK
R-40
R-45
R-50
R-55
R-60
R-65

PLATE 24



VELOCITIES AND CURRENT DIRECTIONS
PLAN E

DISCHARGE 210,000 CFS
TAILWATER 413.0 FT

SCALES IN FEET

PROTOTYPE 600 0 600 1200 1800 2400
MODEL 5 0 5 10 15 20

LEGEND

- 4.2 VELOCITY IN FEET PER SECOND
- VELOCITY LESS THAN 0.5 FEET PER SECOND
- MI 204 RIVER MILES
- STONE FILL DIKE OR REVETMENT
- PROPOSED LEVEE
- EXISTING LEVEE

NOTE: VELOCITIES AND CURRENT DIRECTIONS OBTAINED WITH FLOAT SUBMERGED TO DRAFT OF LOADED BARGE (9.0 FT)
ALL CONTOURS AND ELEVATIONS IN FEET REFERRED TO MSL
NORMAL UPPER POOL VARIES FROM EL 414.0 TO EL 419.0

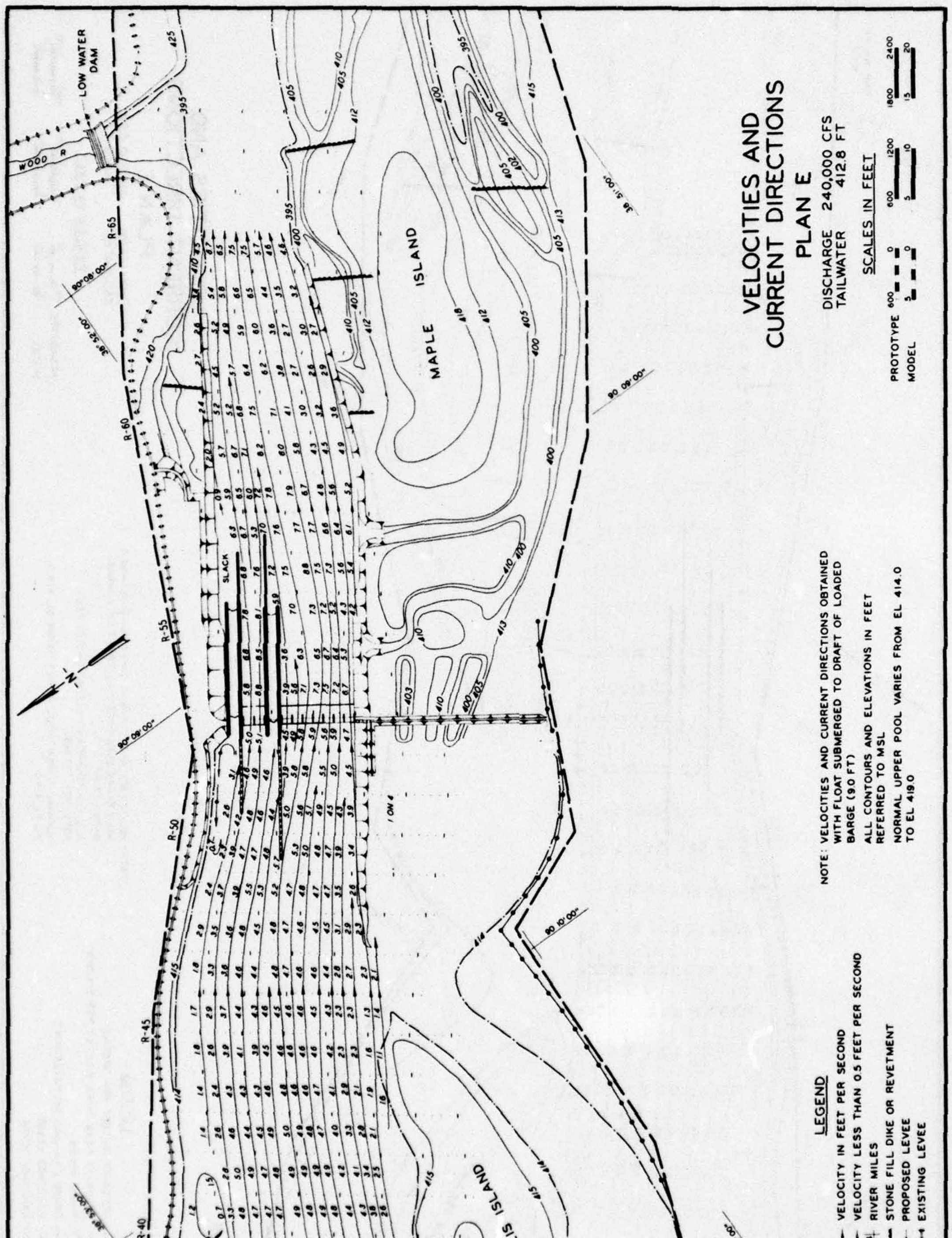
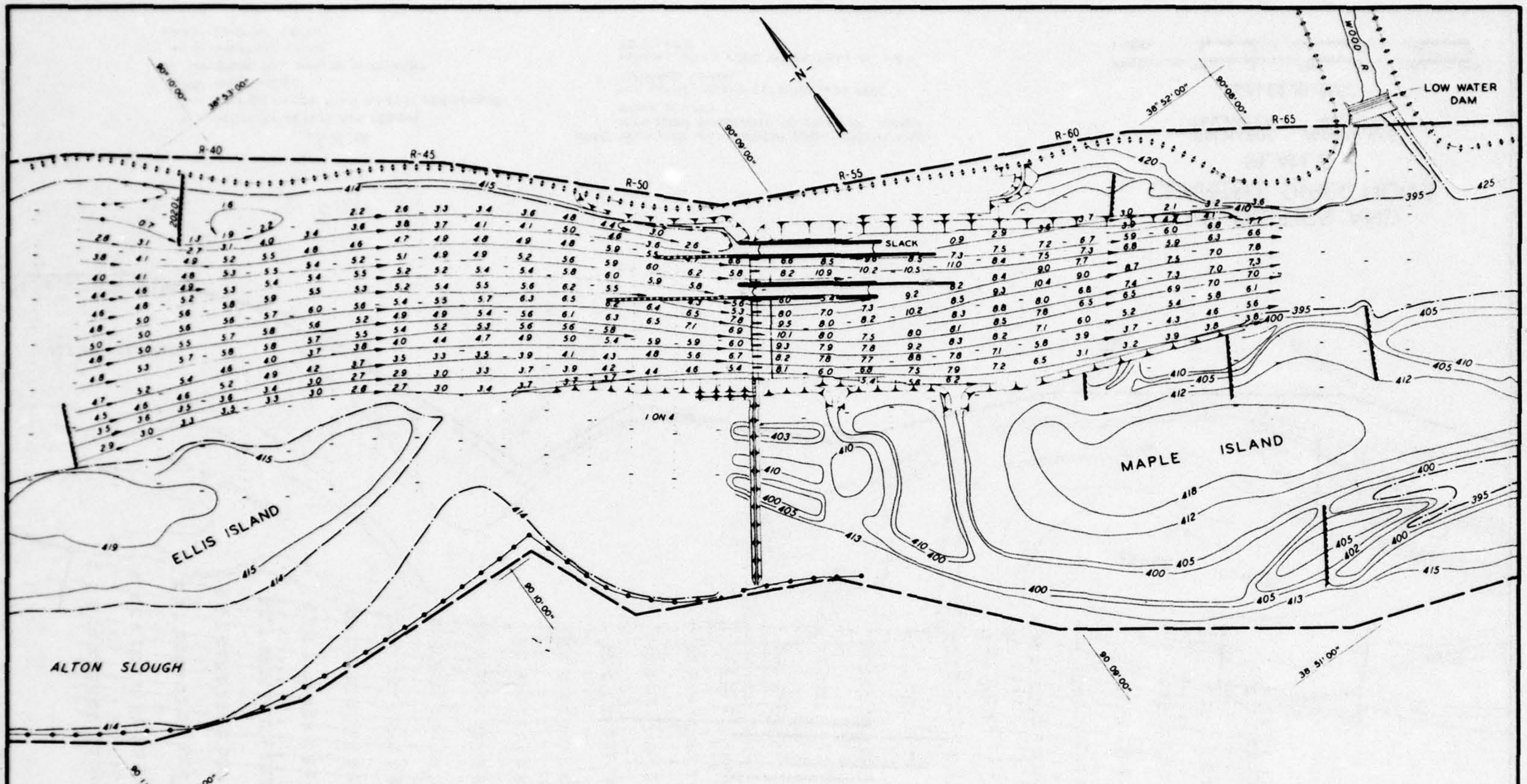

VELOCITIES AND CURRENT DIRECTIONS
PLAN E
DISCHARGE 240,000 CFS
TAILWATER 412.8 FT
SCALES IN FEET
PROTOTYPE 600 0 600 1200 1800 2400
MODEL 5 0 5 10 15 20
NOTE: VELOCITIES AND CURRENT DIRECTIONS OBTAINED WITH FLOAT SUBMERGED TO DRAFT OF LOADED BARGE (9.0 FT)
ALL CONTOURS AND ELEVATIONS IN FEET REFERRED TO MSL
NORMAL UPPER POOL VARIES FROM EL 414.0 TO EL 419.0
LEGEND
4.2 VELOCITY IN FEET PER SECOND
VELOCITY LESS THAN 0.5 FEET PER SECOND
MI 204 RIVER MILES
STONE FILL DIKE OR REVETMENT
PROPOSED LEVEE
EXISTING LEVEE
LOW WATER DAM
WOOD R
MAPLE ISLAND
ELLIS ISLAND
ALTON SLOUGH
SLACK
R-40
R-45
R-50
R-55
R-60
R-65

PLATE 26



VELOCITIES AND CURRENT DIRECTIONS
PLAN E

DISCHARGE 339,000 CFS
TAILWATER 416.0 FT

SCALES IN FEET

PROTOTYPE 600 0 600 1200 1800 2400
MODEL 5 0 5 10 15 20

LEGEND

- 4.2 VELOCITY IN FEET PER SECOND
- • VELOCITY LESS THAN 0.5 FEET PER SECOND
- MI 204 RIVER MILES
- STONE FILL DIKE OR REVETMENT
- PROPOSED LEVEE
- EXISTING LEVEE

NOTE: VELOCITIES AND CURRENT DIRECTIONS OBTAINED WITH FLOAT SUBMERGED TO DRAFT OF LOADED BARGE (9.0 FT)

ALL CONTOURS AND ELEVATIONS IN FEET REFERRED TO MSL

NORMAL UPPER POOL VARIES FROM EL 414.0 TO EL 419.0

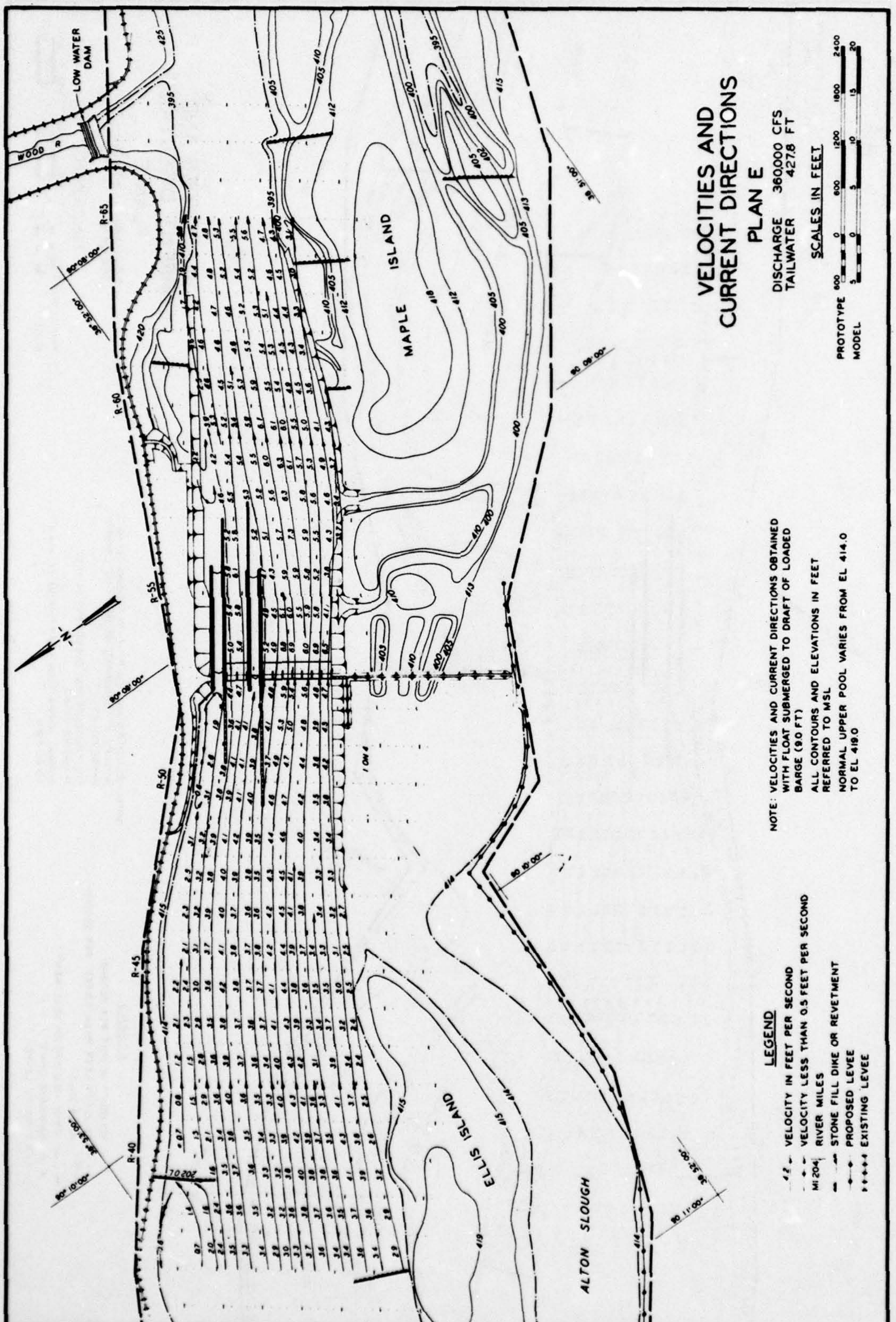

VELOCITIES AND CURRENT DIRECTIONS
PLAN E
DISCHARGE 360,000 CFS
TAILWATER 427.8 FT
SCALES IN FEET
PROTOTYPE
MODEL
NOTE: VELOCITIES AND CURRENT DIRECTIONS OBTAINED WITH FLOAT SUBMERGED TO DRAFT OF LOADED BARGE (9.0 FT)
ALL CONTOURS AND ELEVATIONS IN FEET REFERRED TO MSL
NORMAL UPPER POOL VARIES FROM EL 414.0 TO EL 419.0
LEGEND
VELOCITY IN FEET PER SECOND
VELOCITY LESS THAN 0.5 FEET PER SECOND
RIVER MILES
STONE FILL DIKE OR REVETMENT
PROPOSED LEVEE
EXISTING LEVEE
MAPLE ISLAND
ELLIS ISLAND
ALTON SLOUGH
LOW WATER DAM
WOOD R
R-40
R-45
R-50
R-55
R-60
R-65

PLATE 28

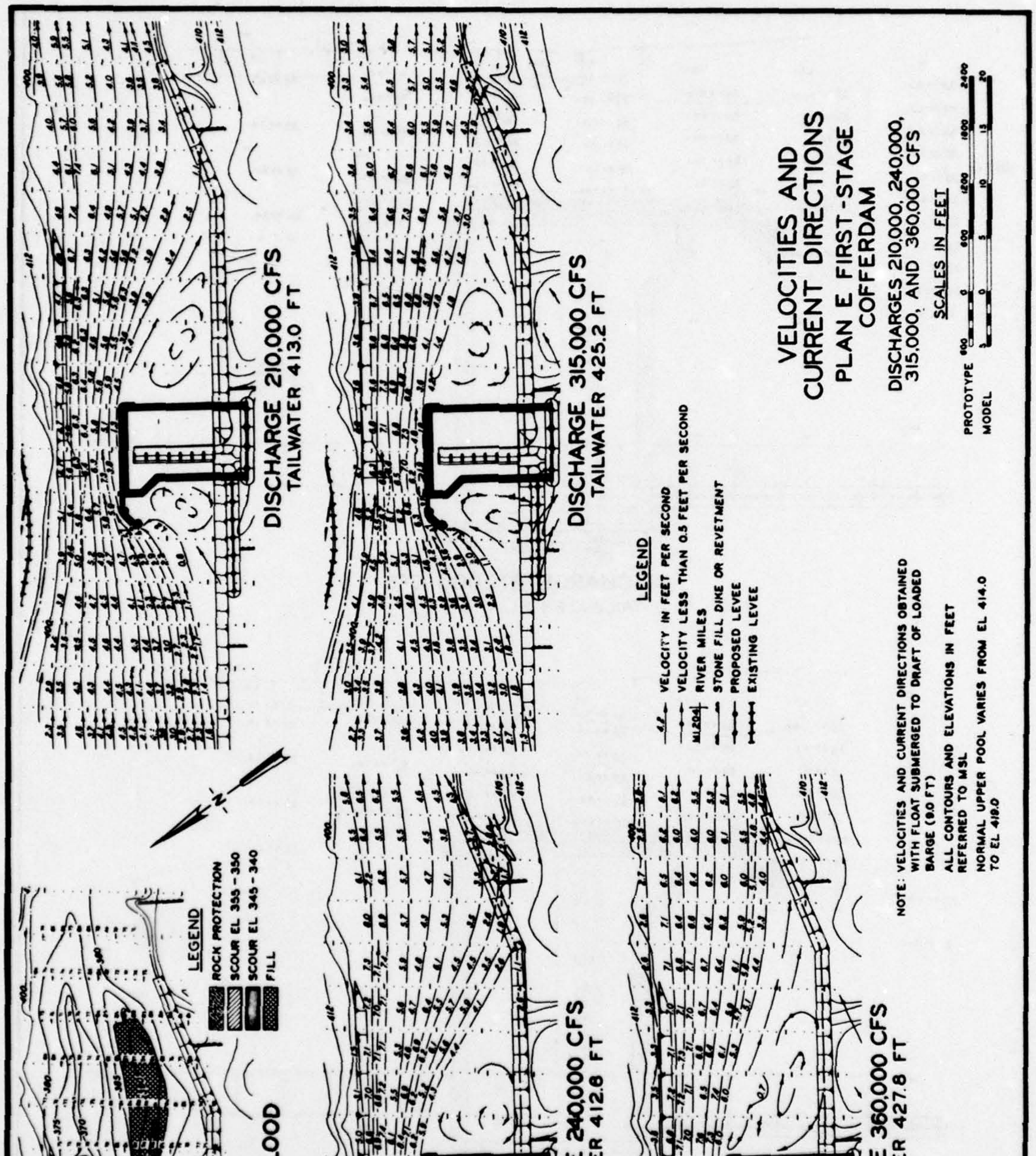
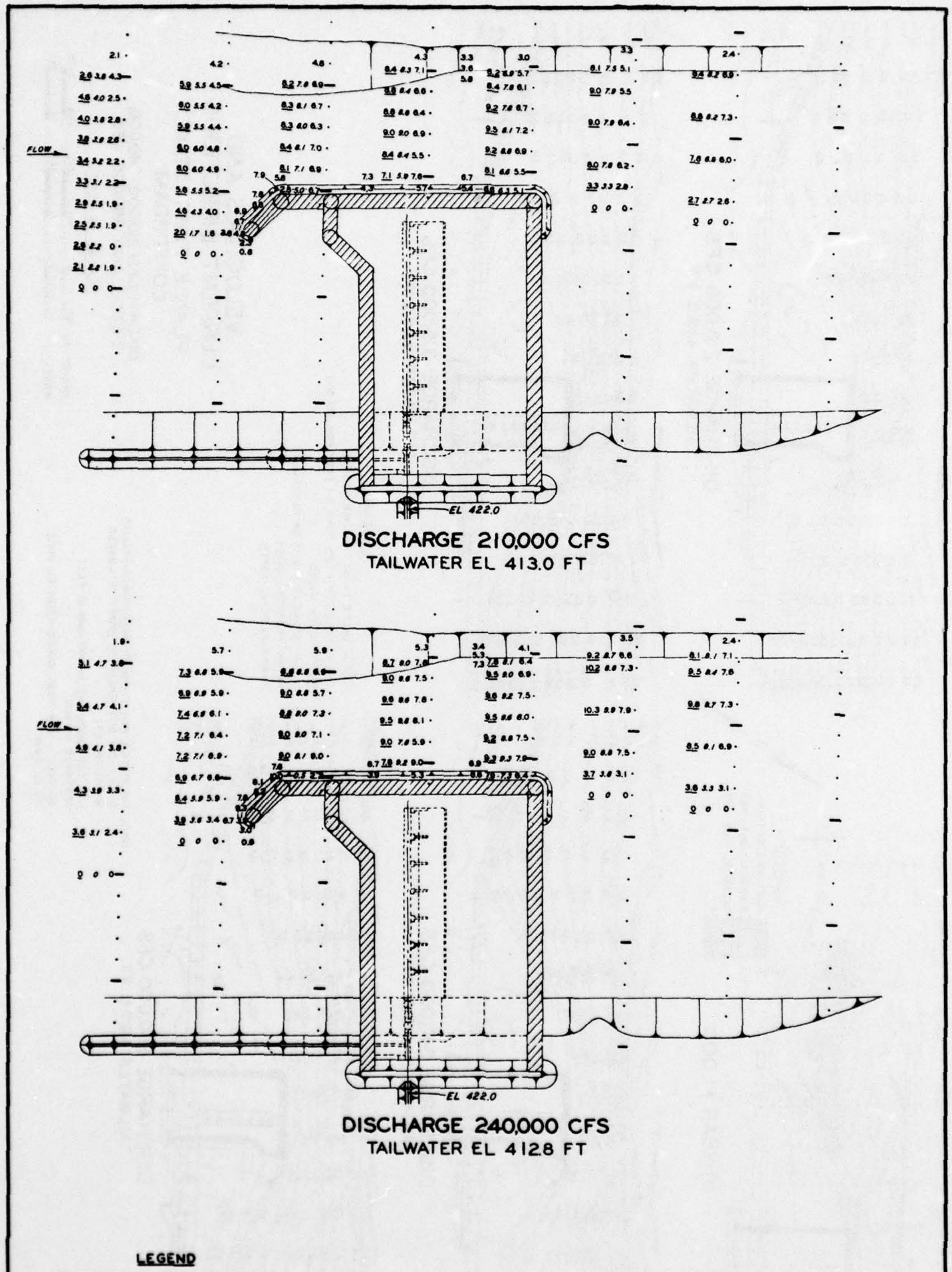
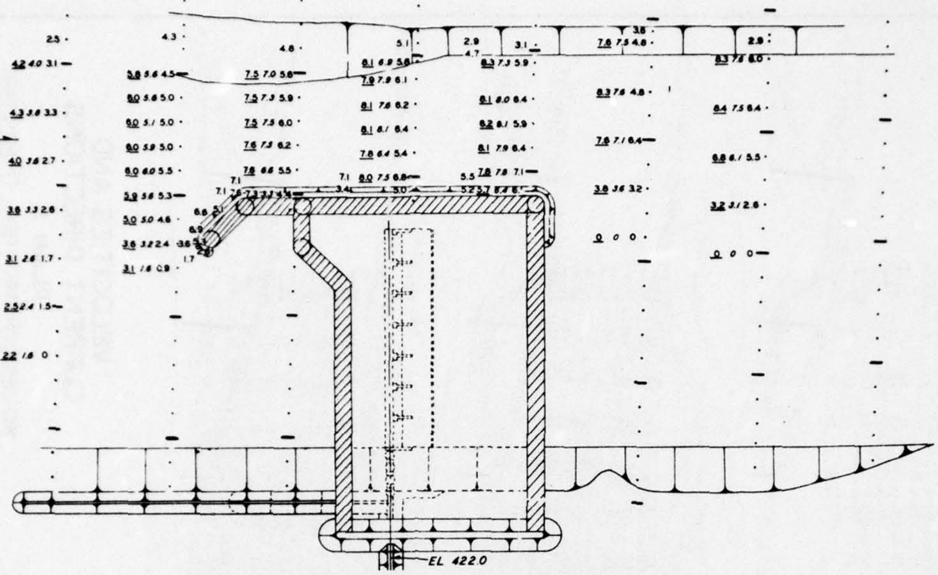
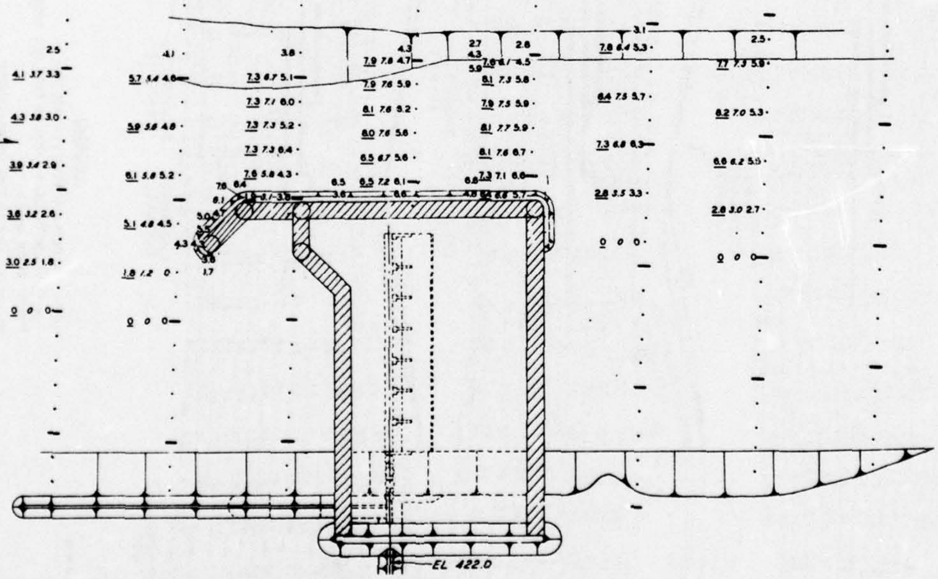

10-YEAR FLOOD
LEGEND
ROCK PROTECTION
SCOUR EL 335 - 350
SCOUR EL 345 - 340
FILL
DISCHARGE 210,000 CFS
TAILWATER 413.0 FT
DISCHARGE 240,000 CFS
TAILWATER 412.8 FT
DISCHARGE 315,000 CFS
TAILWATER 425.2 FT
DISCHARGE 360,000 CFS
TAILWATER 427.8 FT
LEGEND
VELOCITY IN FEET PER SECOND
VELOCITY LESS THAN 0.5 FEET PER SECOND
RIVER MILES
STONE FILL DIKE OR REVETMENT
PROPOSED LEVEE
EXISTING LEVEE
NOTE: VELOCITIES AND CURRENT DIRECTIONS OBTAINED WITH FLOAT SUBMERGED TO DRAFT OF LOADED BARGE (9.0 FT)
ALL CONTOURS AND ELEVATIONS IN FEET REFERRED TO MSL
NORMAL UPPER POOL VARIES FROM EL 414.0 TO EL 418.0
VELOCITIES AND CURRENT DIRECTIONS
PLAN E FIRST-STAGE COFFERDAM
DISCHARGES 210,000, 240,000, 315,000, AND 360,000 CFS
SCALES IN FEET
PROTOTYPE
MODEL

PLATE 29


FLOW
EL 422.0
DISCHARGE 210,000 CFS
TAILWATER EL 413.0 FT
FLOW
EL 422.0
DISCHARGE 240,000 CFS
TAILWATER EL 412.8 FT
LEGEND
4.0 VELOCITY AT 20% OF DEPTH
3.4 VELOCITY AT 60% OF DEPTH
2.7 BOTTOM VELOCITY
SPOT VELOCITIES
PLAN E FIRST - STAGE COFFERDAM
DISCHARGES 210,000 AND 240,000 CFS



DISCHARGE 315,000 CFS
TAILWATER EL 425.2 FT



DISCHARGE 360,000 CFS
TAILWATER EL 427.8 FT

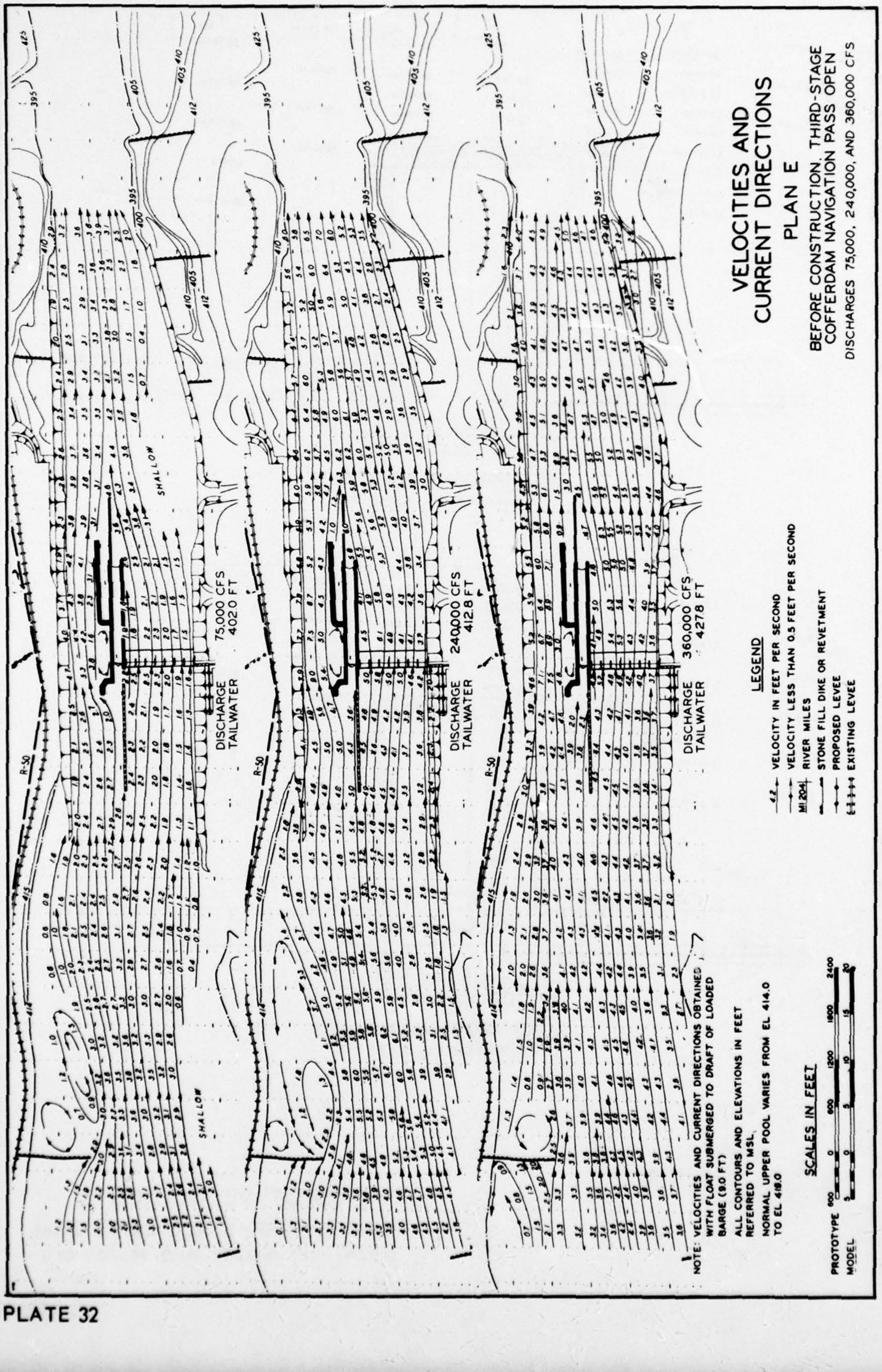
LEGEND

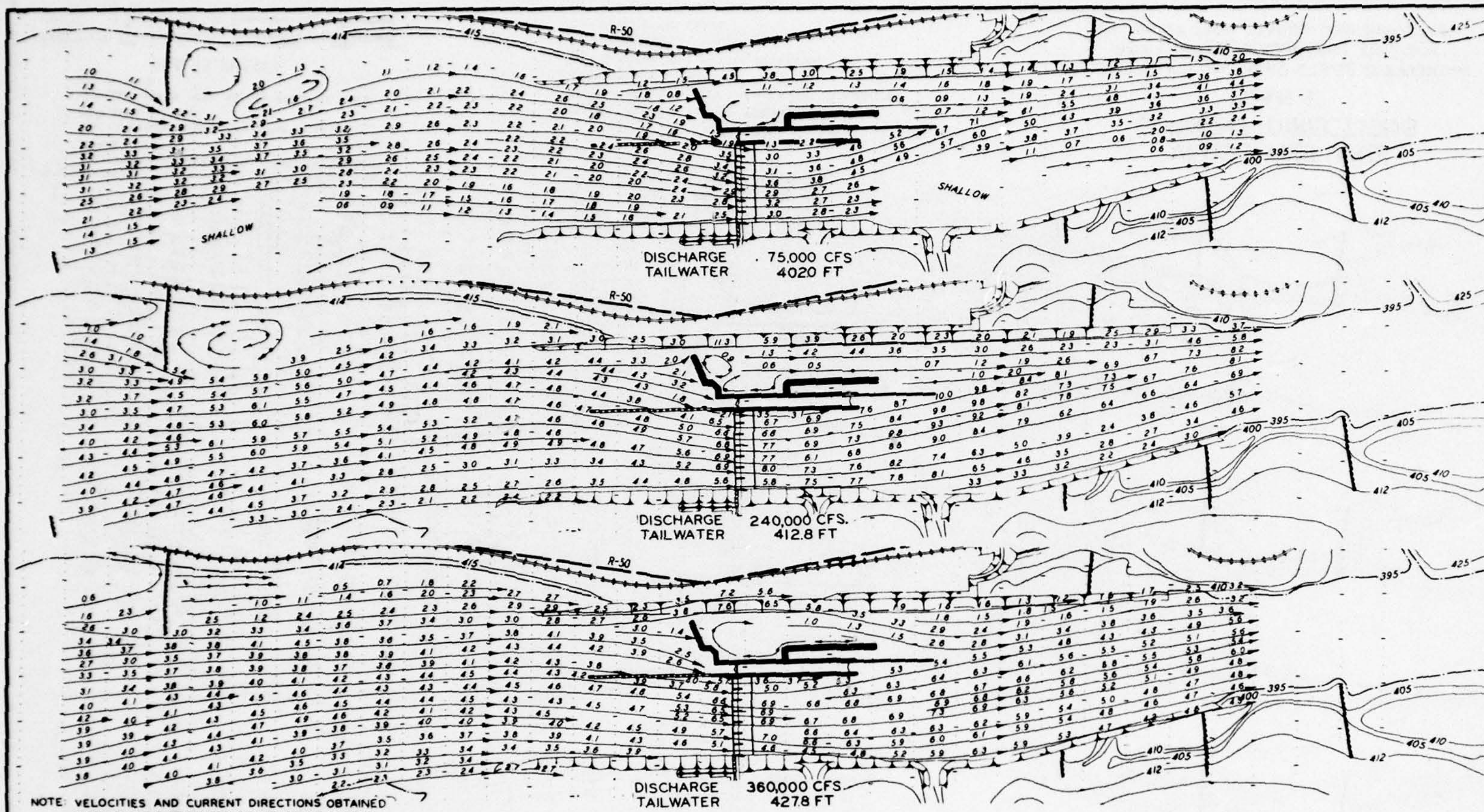
4.0 VELOCITY AT 20% OF DEPTH
3.4 VELOCITY AT 60% OF DEPTH
2.7 BOTTOM VELOCITY

SPOT VELOCITIES

PLAN E FIRST-STAGE COFFERDAM

DISCHARGES 315,000 AND 360,000 CFS


VELOCITIES AND
CURRENT DIRECTIONS
PLAN E
BEFORE CONSTRUCTION, THIRD-STAGE
COFFERDAM NAVIGATION PASS OPEN
DISCHARGES 75,000, 240,000, AND 360,000 CFS
DISCHARGE 75,000 CFS
TAILWATER 402.0 FT
DISCHARGE 240,000 CFS
TAILWATER 412.8 FT
DISCHARGE 360,000 CFS
TAILWATER 427.8 FT
LEGEND
VELOCITY IN FEET PER SECOND
VELOCITY LESS THAN 0.5 FEET PER SECOND
RIVER MILES
STONE FILL DIKE OR REVETMENT
PROPOSED LEVEE
EXISTING LEVEE
NOTE: VELOCITIES AND CURRENT DIRECTIONS OBTAINED WITH FLOAT SUBMERGED TO DRAFT OF LOADED BARGE (9.0 FT)
ALL CONTOURS AND ELEVATIONS IN FEET REFERRED TO MSL.
NORMAL UPPER POOL VARIES FROM EL 414.0 TO EL 418.0
SCALES IN FEET
PROTOTYPE
MODEL
SHALLOW
R-50



NOTE: VELOCITIES AND CURRENT DIRECTIONS OBTAINED WITH FLOAT SUBMERGED TO DRAFT OF LOADED BARGE (9.0 FT)

ALL CONTOURS AND ELEVATIONS IN FEET REFERRED TO MSL

NORMAL UPPER POOL VARIES FROM EL 414.0 TO EL 419.0

SCALES IN FEET

PROTOTYPE 600 0 600 1200 1800 2400

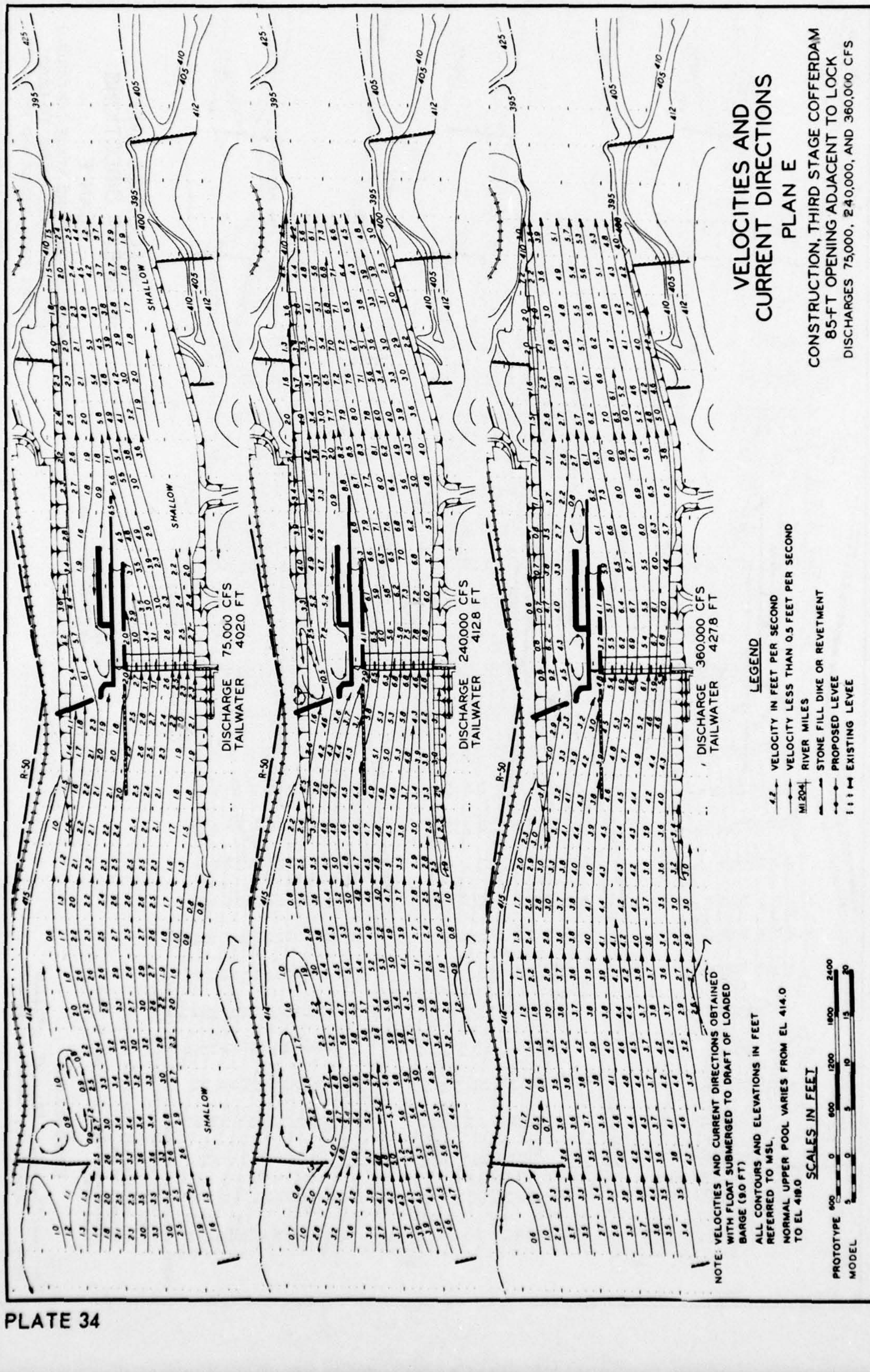
MODEL 5 0 5 10 15 20

LEGEND

- 4.2 VELOCITY IN FEET PER SECOND
- VELOCITY LESS THAN 0.5 FEET PER SECOND
- MI 204 RIVER MILES
- STONE FILL DIKE OR REVETMENT
- PROPOSED LEVEE
- EXISTING LEVEE

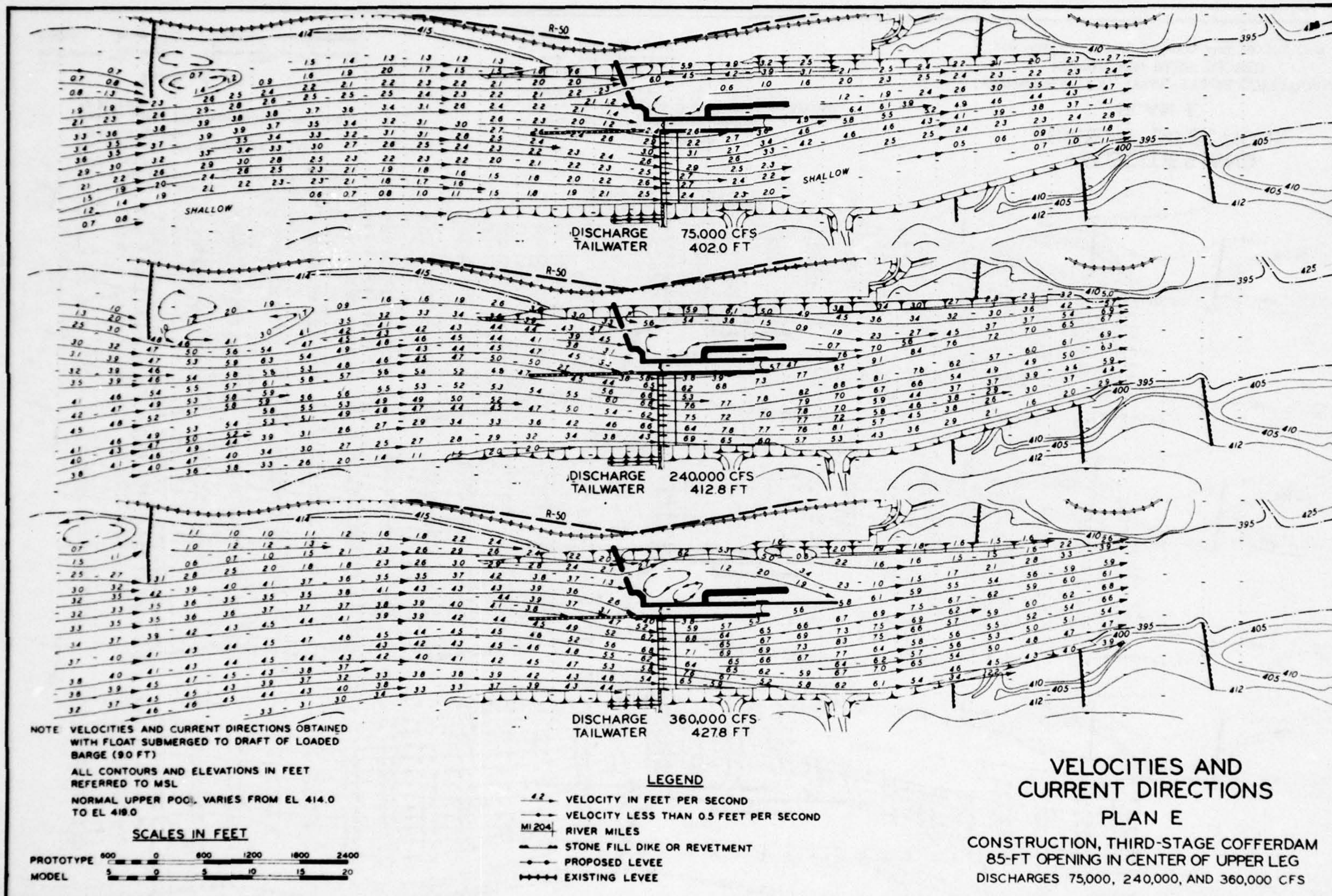
VELOCITIES AND CURRENT DIRECTIONS
PLAN E

CONSTRUCTION, THIRD STAGE COFFERDAM
80-FT OPENING ADJACENT TO BANK
DISCHARGES 75,000, 240,000, AND 360,000 CFS

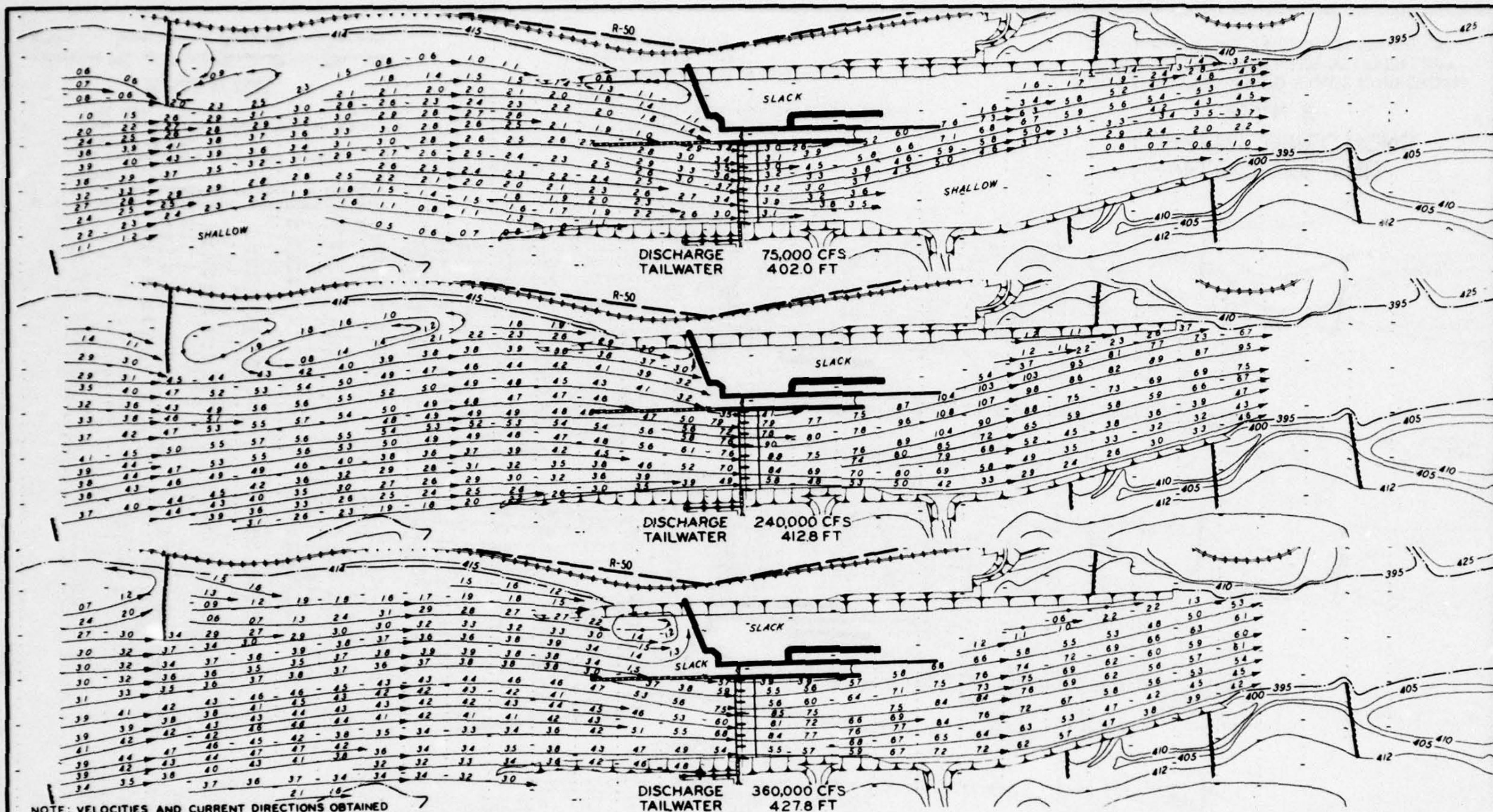


VELOCITIES AND
CURRENT DIRECTIONS
PLAN E
CONSTRUCTION, THIRD STAGE COFFERDAM
85-FT OPENING ADJACENT TO LOCK
DISCHARGES 75,000, 240,000, AND 360,000 CFS
DISCHARGE 75,000 CFS
TAILWATER 4020 FT
DISCHARGE 240000 CFS
TAILWATER 4128 FT
DISCHARGE 360000 CFS
TAILWATER 4278 FT
SHALLOW
R-50
LEGEND
4.2 VELOCITY IN FEET PER SECOND
VELOCITY LESS THAN 0.5 FEET PER SECOND
MI 204 RIVER MILES
STONE FILL DIKE OR REVETMENT
PROPOSED LEVEE
EXISTING LEVEE
NOTE: VELOCITIES AND CURRENT DIRECTIONS OBTAINED WITH FLOAT SUBMERGED TO DRAFT OF LOADED BARGE (9.0 FT)
ALL CONTOURS AND ELEVATIONS IN FEET REFERRED TO MSL.
NORMAL UPPER POOL VARIES FROM EL 414.0 TO EL 419.0
SCALES IN FEET
PROTOTYPE 600 0 600 1200 1800 2400
MODEL 5 0 5 10 15 20

PLATE 34



VELOCITIES AND CURRENT DIRECTIONS
PLAN E
CONSTRUCTION, THIRD-STAGE COFFERDAM
85-FT OPENING IN CENTER OF UPPER LEG
DISCHARGES 75,000, 240,000, AND 360,000 CFS



NOTE: VELOCITIES AND CURRENT DIRECTIONS OBTAINED WITH FLOAT SUBMERGED TO DRAFT OF LOADED BARGE (9.0 FT)

ALL CONTOURS AND ELEVATIONS IN FEET REFERRED TO MSL.

NORMAL UPPER POOL VARIES FROM EL 414.0 TO EL 419.0

SCALES IN FEET

PROTOTYPE 600 0 600 1200 1800 2400

MODEL 5 0 5 10 15 20

LEGEND

42 VELOCITY IN FEET PER SECOND

VELOCITY LESS THAN 0.5 FEET PER SECOND

MI 204 RIVER MILES

STONE FILL DIKE OR REVETMENT

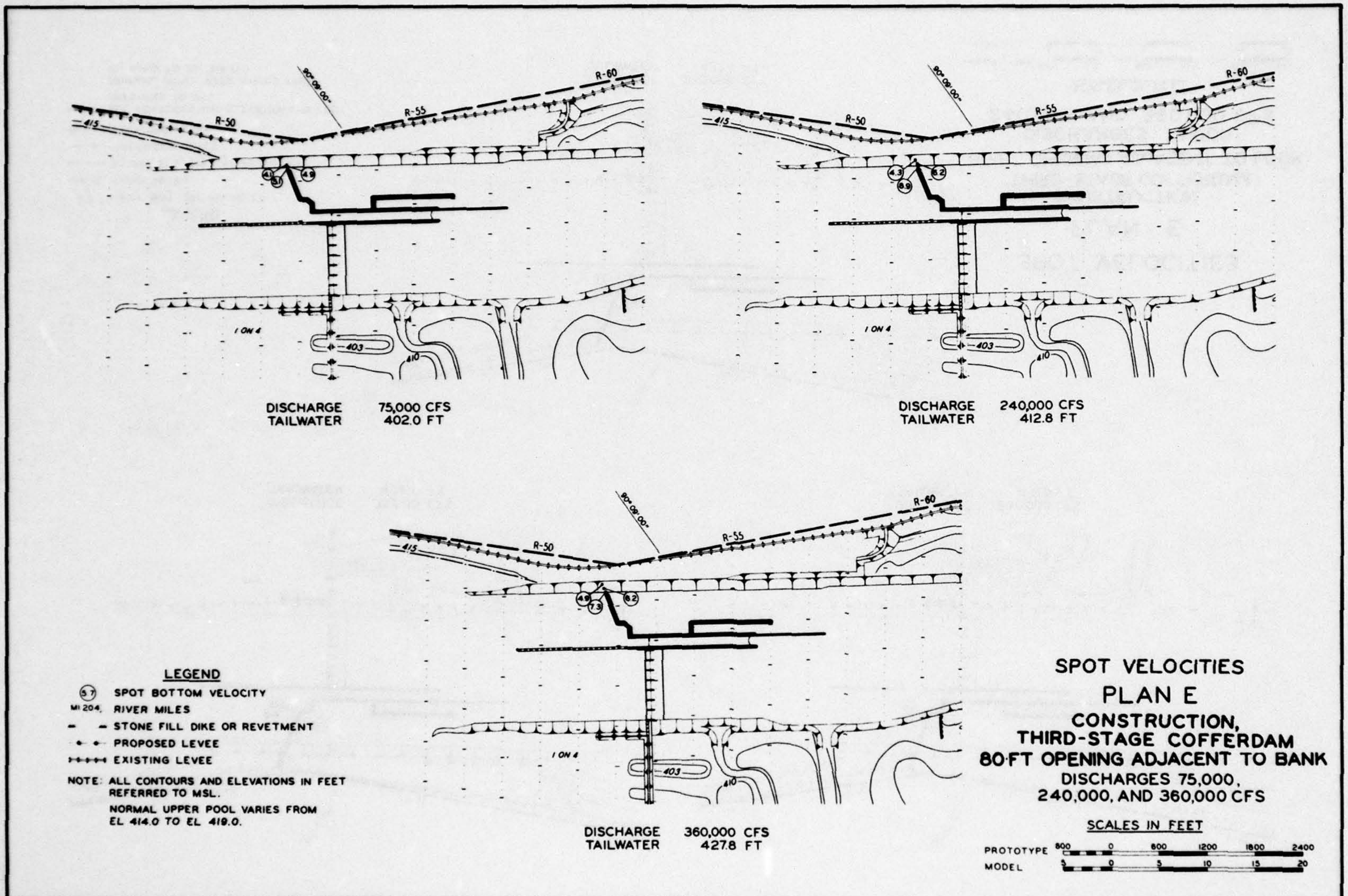
PROPOSED LEVEE

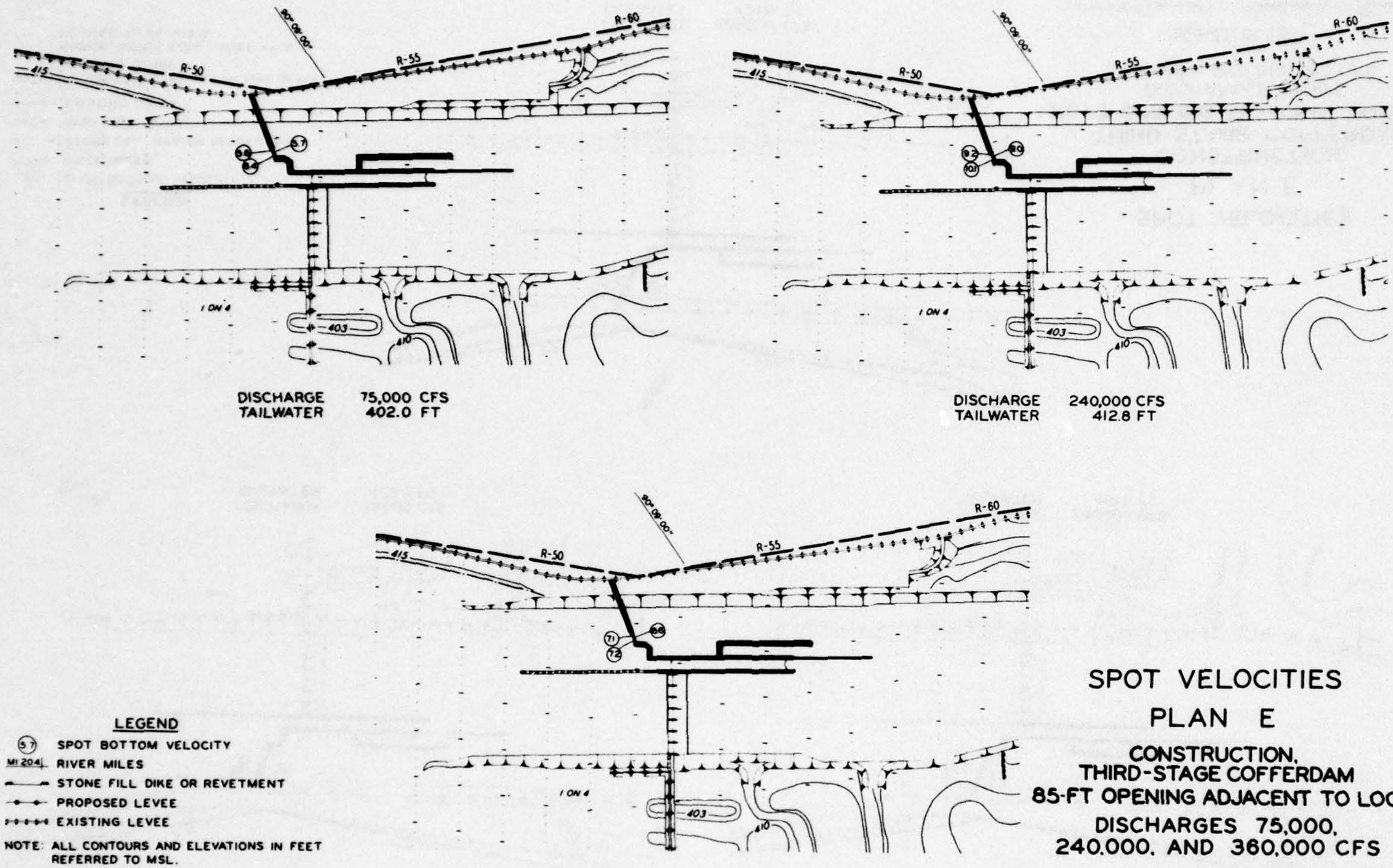
EXISTING LEVEE

VELOCITIES AND CURRENT DIRECTIONS

PLAN E

CONSTRUCTION, THIRD-STAGE COFFERDAM
NAVIGATION PASS CLOSED
DISCHARGES 75,000, 240,000, AND 360,000 CFS


R-50
R-55
R-60
90° 09' 00"
415
403
410
1 ON 4
DISCHARGE 75,000 CFS
TAILWATER 402.0 FT
DISCHARGE 240,000 CFS
TAILWATER 412.8 FT
DISCHARGE 360,000 CFS
TAILWATER 427.8 FT
LEGEND
5.7 SPOT BOTTOM VELOCITY
MI 204. RIVER MILES
STONE FILL DIKE OR REVETMENT
PROPOSED LEVEE
EXISTING LEVEE
NOTE: ALL CONTOURS AND ELEVATIONS IN FEET REFERRED TO MSL.
NORMAL UPPER POOL VARIES FROM EL 414.0 TO EL 419.0.
SPOT VELOCITIES
PLAN E
CONSTRUCTION,
THIRD-STAGE COFFERDAM
80-FT OPENING ADJACENT TO BANK
DISCHARGES 75,000,
240,000, AND 360,000 CFS
SCALES IN FEET
PROTOTYPE 800 0 600 1200 1800 2400
MODEL 5 0 5 10 15 20



DISCHARGE 75,000 CFS
TAILWATER 402.0 FT

DISCHARGE 240,000 CFS
TAILWATER 412.8 FT

DISCHARGE 360,000 CFS
TAILWATER 427.8 FT

LEGEND

- (5.7) SPOT BOTTOM VELOCITY
- MI 204 RIVER MILES
- STONE FILL DIKE OR REVETMENT
- PROPOSED LEVEE
- EXISTING LEVEE

NOTE: ALL CONTOURS AND ELEVATIONS IN FEET REFERRED TO MSL.

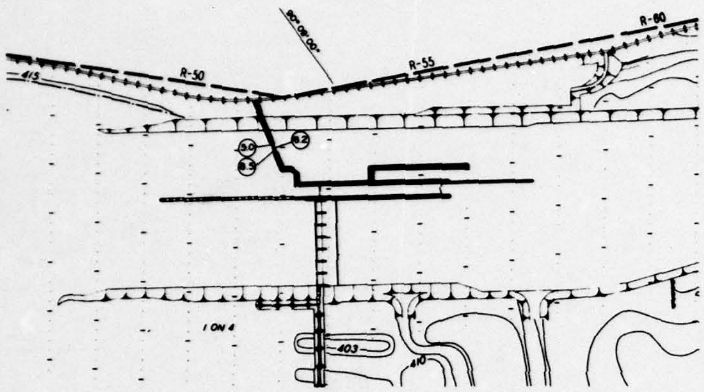
NORMAL UPPER POOL VARIES FROM EL 414.0 TO EL 419.0.

SPOT VELOCITIES
PLAN E

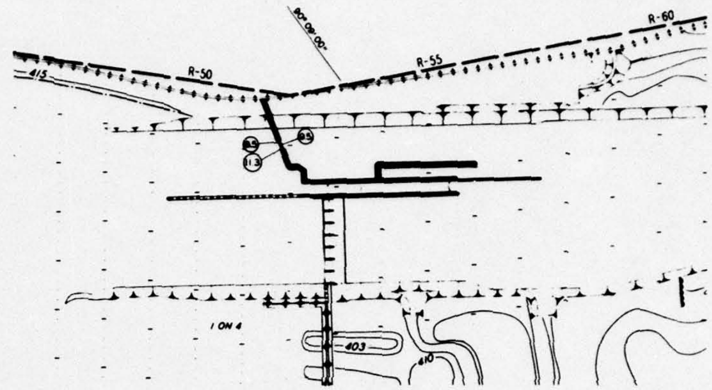
CONSTRUCTION,
THIRD-STAGE COFFERDAM
85-FT OPENING ADJACENT TO LOCK
DISCHARGES 75,000,
240,000, AND 360,000 CFS

SCALES IN FEET

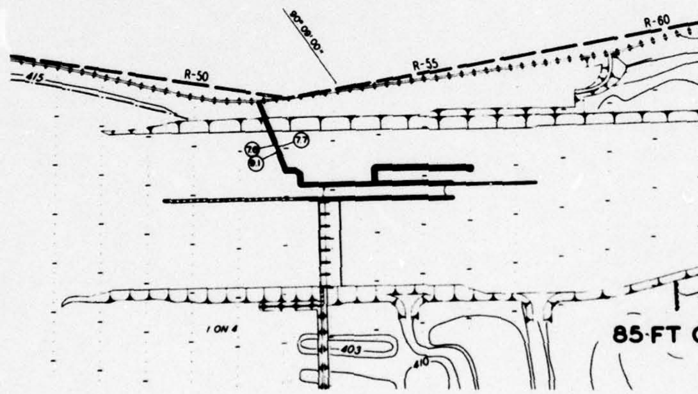
PROTOTYPE 600 0 600 1200 1800 2400
MODEL 5 0 5 10 15 20



DISCHARGE 75,000 CFS
TAILWATER 402.0 FT



DISCHARGE 240,000 CFS
TAILWATER 412.8 FT



DISCHARGE 360,000 CFS
TAILWATER 427.8 FT

LEGEND

- (5.7) SPOT BOTTOM VELOCITY
- MI 204 RIVER MILES
- STONE FILL DIKE OR REVETMENT
- PROPOSED LEVEE
- EXISTING LEVEE

NOTE: ALL CONTOURS AND ELEVATIONS IN FEET REFERRED TO MSL.
NORMAL UPPER POOL VARIES FROM EL 414.0 TO EL 419.0.

SPOT VELOCITIES
PLAN E
CONSTRUCTION, THIRD-STAGE COFFERDAM 85-FT OPENING IN CENTER OF UPPER LEG DISCHARGES 75,000, 240,000, AND 360,000 CFS

SCALES IN FEET

PROTOTYPE	600	0	600	1200	1800	2400
MODEL	5	0	5	10	15	20

In accordance with letter from DAEN-RDC, DAEN-ASI dated 22 July 1977, Subject: Facsimile Catalog Cards for Laboratory Technical Publications, a facsimile catalog card in Library of Congress MARC format is reproduced below.

Shows, Louis J

Navigation conditions at Locks and Dam 26, Mississippi River; hydraulic model investigation / by Louis J. Shows, John J. Franco. Vicksburg, Miss. : U. S. Waterways Experiment Station ; Springfield, Va. : available from National Technical Information Service, 1979.

63, [24] p., 39 leaves of plates : ill. ; 27 cm. (Technical report - U. S. Army Engineer Waterways Experiment Station ; HL-79-19)

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1. Fixed-bed models. 2. Hydraulic models. 3. Lock and Dam No. 26, Mississippi River. 4. Locks (Waterways). 5. Mississippi River. 6. Navigation conditions. I. Franco, John J., joint author. II. United States. Army. Corps of Engineers. St. Louis District. III. Series: United States. Waterways Experiment Station, Vicksburg, Miss. Technical report ; HL-79-19.

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